Northern Golden Gate Estates Flowway Restoration Project

Collier County

May 9, 2013

Plan Design Enable

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Executive Summary

The North Golden Gate Estates Flowway Restoration Project (NGGEFRP) has been developed using an integrated surface water/groundwater model that was used to evaluate the affects of implementing different components of the project. Development of the NGGEFRP started with a review of data collected in previous studies and an evaluation of wetland function based on previous studies.

Subsequent tasks included development of a local scale MIKE SHE/MIKE 11 model from the model developed for the Collier County Watershed Management Plan (CCWMP), and an analysis of alternative implementation configurations.

Task 1. Enhance Database Developed in Phase I

Atkins performed three evaluations during this task. The first evaluation involved the use of GIS tools to define sub-basins within the study area and compare the newly developed sub-basins against the sub-basins developed with older topographic data for the Southwest Florida Feasibility Study.

The results indicated that the SWFFS basins and the basins defined in the task are similar indicating that the basins are generally appropriate for modeling purposes. The results indicate that two basins, including the North Belle Meade area, may require modification; however, the differences are unlike to affect the overall hydrology of the study area.

A second evaluation included review and ground-truthing of wetland maps developed during previous studies. In general it was determined that the previously developed wetland maps were not consistent with existing conditions and omit many wetland areas that were created during residential development. Revised wetland maps were created to guide the evaluation of restoration activities and to better define land uses in the modeling effort

A final evaluation considered the change in wetland communities within the study area. It was found that more than 8,200 acres of hydric flatwood and cypress wetlands have been lost in the study area. These areas have been converted to urban land uses, or the hydrology has changed such that the land cover is now consistent with upland mesic communities. This information was used to provide initial guidance on potential restoration activities.

Task 2. Develop a Local Scale Model

In this portion of the project, several sub-tasks were completed. These included development of a local scale model from the regional 7-layer MIKE She/MIKE 11 model developed for the CCWMP and accepted by the South Florida Water Management District (SFWMD); the inclusion of addition hydraulic features and land use updates; and calibration of the local scale model.

Sub-tasks 1 and 2 were completed concurrently and included the following elements.

- Define the local scale model domain and grid size;
- Define land use input data for the local scale model;
- Define topography for the local scale model;
- Reduce the MIKE 11 River network to correspond to the new domain;
- Extract boundary conditions from the CC ECM; and
- Run initial simulation and compare results against the CC ECM
- Add additional hydraulic features

A comparison of the initial local scale model to the 7-layer CCWMP model indicated that the local scale model produced very similar or better results than the 7-layer CCWMP model. The comparison also indicated that the predicted peak stages in the canals are often one (1) or more feet lower in the local scale model than the larger scale CCWMP model and more closely match the observed data. This is likely related to the more refined topography associated with 500-ft grid and suggests that more water is stored on the ground surface. The updated land use data may also reduce runoff since some lands that were simulated as low density urban in the CCWMP model are now accurately represented as natural areas in the local scale model.

The final sub-task was to calibrate the local scale model. During review of available observation data, several monitoring wells were identified for use in the local scale model domain that were not utilized during calibration of the CCWMP model. A total of nine simulations were completed to calibrate the local scale model.

The greatest challenge to improve the model calibration was to establish appropriate boundary conditions. In the eastern and southern portions of the model domain, the data extracted from the CCWMP was determined to be insufficient to represent the boundary conditions. Subsequently, monitoring data from groundwater wells located outside the model domain, but near the boundary was applied as a new boundary condition. In several cases, the data from several wells was averaged to derive the boundary condition. In cases where observation data did not cover the entire simulation period, data was averaged on a daily basis to define a typical Julian year condition. This condition was then repeated for each year of the simulation period.

The following conclusions were drawn about the usefulness of the model to evaluate proposed projects.

- The calibration statistics for the local scale model are very similar to; or better than the statistics for the CCWMP model. In addition, the predicted hydroperiod map of the study area is considered to be very good. These results indicate that the model is an appropriate tool to evaluate potential projects in the flowway.

- In the center of the model area the results indicate a general over-prediction of head elevation in the Water Table aquifer of approximately 5.5 inches. This may influence the hydroperiod in wetland areas and was considered when evaluating potential project benefits.

- The water surface elevations in the canals are generally well calibrated and control the groundwater head elevation. The wet season stage is managed by control structures and influences the hydroperiod of adjacent lands. This also was considered when evaluating potential project benefits.

Task 3. Alternatives Analysis and Preparation of Preliminary Design Drawings

The final task of the project was to evaluate alternative water management scenarios within the study area and prepare preliminary design drawings and a cost estimate for each recommended element. Four scenarios were defined and evaluated during this process. A brief description of each scenario is provided below.

Scenario 1. This alternative focuses on the North Belle Meade area between I-75 and the Golden Gate Canal in the headwaters of the Rookery Bay watershed. This scenario considers the diversion of water from the Golden Gate Main Canal into a spreader system that will direct water into the North Belle Meade area north of I-75. Two options were considered for this scenario. Option 1 assumed an 800 cfs pump station to divert water. Option 2 considered a 400 cfs pump station.

Scenario 2. This alternative is located in the portion of study area north of Oil Well Rd, and south of Immokalee Rd between the Golden Gate and Faka Union Canals. This scenario focuses on adding cross-drains under existing roads to improve the interconnection of wetlands with the goal of rehydrating and extending the hydroperiod of currently drained wetlands and potentially increasing groundwater recharge.

Scenario 3. This alternative is similar to Scenario 2 in that it included the inclusion of cross-drains for the purpose of rehydrating wetland areas. This scenario was applied to the area south of Oil Well Rd and extending to I-75 in the south and located between the Golden Gate/Miller Canal and the Faka Union Canal. This scenario also included a small area west of the Miller Canal on either side of Golden Gate Blvd.

Scenario 4. This scenario combines those features of Scenarios 1, 2, and 3 that were determined to provide a hydrologic benefit and was completed in order to predict the cumulative effect of all improvements that may be recommended.

Scenario 1 Results

The results of Scenario 1 indicated that both options would provide a positive benefit in reducing the volume of flow to Naples Bay. However, in both cases, the model results showed that the diverted water was not stored in the North Belle Meade area for a long period of time. Generally, the inflows to the wetland area equaled the outflows after approximately only a few weeks of pumping. The water was predicted to flow into the Henderson Canal or the Miller Canal and would likely flow to the Rookery Bay or Ten Thousand Islands estuaries.

The following conclusions were drawn from the analysis completed for Scenario 1.

- Outflows to Naples Bay may be reduced by an average of three (3) percent per month during the wet season if an 800 cfs pump station is utilized. Outflows may be reduced by 2.5 percent per wet season month with a 400 cfs pump station.
- The North Belle Meade area has limited storage and infiltration capacity such that outflows closely match inflows after only a few weeks of operation.
- Using an 800 cfs pump station may double the predicted flows to Henderson Creek. The CCWMP identified a potential flow surplus to Rookery Bay from Henderson Creek and this could increase the predicted flow surplus during the period from June – September unless other projects are implemented to divert or store the additional flow. A 400 cfs pump would likely increase flows to Henderson Creek by more than one-third.
- The increased flows to Henderson Creek would likely contribute to the reduction of a flow deficit to the Rookery Bay Estuary identified in the CCWMP. This would occur during the months of October and November.
- The volume of water in the Miller Canal is increased; however, the pumping rates for the proposed Miller Canal Pump station remain well below the planned capacity of 1,250 cfs...

The results of the Scenario 1 analysis suggest that the use of a diversion system that features an 800 cfs pump station is viable if additional projects are implemented to mitigate the increased flows, and minimize or prevent an increase in flow to the Rookery Bay Estuary during the early months of the wet season. However, the 800 cfs pump station configuration considered in this evaluation may not be cost effective since the pump rarely operates at maximum capacity.

Scenario 2 Results

In this evaluation, a total of 35 additional culverts were considered for inclusion in the area north of Oil Well Rd. These cross-drains were placed at locations where the topographic data suggested that overland flow had once occurred and would allow water to move under the road into other wetland areas rather than drain to the canal network.

The model results indicated that flow rates in the Golden Gate and Faka Union Canals were lower during the simulation period. This suggests that more water is stored in the wetland network. However, the results also indicated that the inclusion of additional cross-drains tended to lower the water surface and reduce the hydroperiod in several high quality wetlands in the area.

The following conclusions were drawn from the analysis completed for Scenario 2.

- The addition of the proposed cross-drains contributes to increased infiltration and leads to a slight reduction in predicted flows in the Golden Gate and Faka Union Canals.
- Model results also suggest that the addition of the proposed cross-drains may have the unintended consequence of draining existing high quality wetland areas.

The results of the Scenario 2 analysis indicate that adding cross-drains in the areas around the Panther Walk and Winchester Head wetland areas may be detrimental to these preserved wetland systems. Collier County should use caution when deciding how to proceed in the area north of Oil Well Rd. Many of the cross-drains considered in this analysis were not evaluated in Scenario 4.

Scenario 3 Results

In this evaluation, a total of 51 additional culverts were considered for inclusion in the area south of Oil Well Rd and north of I-75. These cross-drains were placed at locations where the topographic data suggested that overland flow had once occurred and would allow water to move under the road into other wetland areas rather than drain to the canal network.

In the Miller Canal, the model results indicate that there is a slight increase in the flow volume throughout the year. The increase averages less than one (1) percent of total flow volume during any particular month. This is likely the result of adding cross-drains to the streets north of 8th Avenue NE. These cross-drains divert water that currently drains to the Golden Gate Canal, south toward the Miller Canal.

In the Faka Union Canal, the results indicate that the proposed cross-drains contribute to an overall reduction in wet season flows, and a slight increase in early dry season flows. These results are consistent with increased infiltration and later baseflow to the canal network.

The results of the Scenario 3 simulation indicate that there is a decrease in hydroperiod of approximately 45 days in the area immediate south of CR858 (Oil Well Rd), east of Everglades Blvd. The results also show on increase in hydroperiod ranging from 5 to 40 days east of Everglades Blvd in the area between 20th Avenue NE and 2nd Avenue SE.

The area of decreased hydroperiod demonstrates a detrimental impact to existing wetland area; whereas, this area of increased hydroperiod may increase recharge within the Collier County wellfield area.

The following conclusions were drawn from the analysis completed for Scenario 3.

- The addition of the proposed cross-drains in the Scenario 3 contributes to a rise in seasonal average and seasonal high groundwater elevation near the County well fields. This will likely result in increased recharge to the well field.
- The addition of the proposed cross-drains appears to divert water from the Golden Gate Canal toward the Miller Canal near 8th Avenue NE, west of Everglades Blvd. This would contribute to slight reduction in flow to Naples Bay.
- There is no perceived benefit to adding cross-drains in the area west of the Miller and Golden Gate Canals along 16th Street NE.
- Model results suggest that homes constructed on relatively low pads may have their septic leach fields affected by the change in the seasonal high groundwater elevation. This is particularly relevant near 8th Avenue NE, west of Everglades Blvd.

The results of the Scenario 3 analysis suggest that adding cross-drains would provide the overall benefit of expanded wetland areas and increases in groundwater elevations near the county wellfield. However, the County would have to consider the consequences to several homes that are constructed on low pads and may be affected by changes in groundwater elevation.

Cross-drains in areas showing a significant decrease in hydroperiod or where there is no perceived benefit were not considered in the Scenario 4 analysis.

Scenario 4 Results

Several different model runs were completed for Scenario 4. Each of the simulations considered the removal of proposed cross-drains that appeared to provide negative or no hydrologic benefit. The final Combined Scenario considers the interaction of the system with the following improvements:

- Scenario 1, Option 1 (800 cfs pump) of the North Belle Meade Spreader Swale
- Many of the cross-drains proposed in Scenario 2 were removed from the model. These include all proposed cross-drains in the Panther Walk area and several in the Winchester Head Area.
- Several of the proposed cross-connects described in Scenario 3 were removed from this scenario. These include those immediately south of Oil Well Rd and several in the southern most part of the study area.

The results of this analysis are very similar to the Scenario 1, Option 1 results throughout the simulation period. However, the Scenario 4 results generally predict slightly less flow throughout the year. Compared to the Scenario 1, Option 1 results, the greatest additional reductions in flow occur in July, November, and December. The additional reduction in flow in July can be attributed to the cross-drains in the Northern Golden Gate Estates area. The cross-drains and diversion structure do not effect water levels in the Golden Gate Canal during the dry season, but do contribute to an overall reduced volume of runoff discharging from the Golden Gate Canal and provide an opportunity for increased infiltration. Reduced flows in November and December are attributable to additional storage created by lower water levels in the Golden Gate Canal between the GG-3 and GG-4 structures during the wet season. The Scenario 4 result predicts that the water level between these structures will be approximately six (6) inches lower at times during the wet season as a result of the diversion pumping. This means that there is more storage available upstream of the GG-3 structure and subsequently contributes to lower total discharge volumes later in the year.

The following conclusions were drawn from the analysis completed for Scenario 4.

- There is an overall improvement in the reduction of flows to Naples Bay resulting from the implementation of the North Belle Meade Spreader project.
- Increased flows to Henderson Creek resulting from the spreader system may need to be mitigated so that there is no increase in flows to Rookery Bay. The South I-75 Spreader Swale and the Henderson Creek off-line storage projects indentified in the CCWMP may be able to offset the additional flow.
- The increased flow to the Miller Canal resulting from the North Belle Meade Spreader does not impact the overall conveyance capacity or the drainage swales that discharge to the Miller Canal. The results indicate that the enhanced flowway will not adversely impact the delivery of water, or the overall goals of the PSRP.
- Adding cross-drains north of Oil Well Rd. may be detrimental to existing wetland areas. It is recommended that zero cross-drains be added in the Panther Walk area.
- Adding cross-drains south of Oil Well Rd. is predicted to increase groundwater elevations near the junction of the Golden Gate and Miller Canals. This may be beneficial by providing additional recharge to the County Well field.
- There is no perceived benefit to adding cross-drains in the area west of the Miller/Golden Gate Canal along 16th Street NE.
- Model results suggest that homes constructed on relatively low pads may have their septic leach fields affected by the change in the seasonal high groundwater elevation. This is particularly relevant near 8th Avenue NE, west of Everglades Blvd.

The results of the Scenario 4 analysis suggest that the project as a whole will partially meet the goals of diverting water from the Golden Gate Canal, increasing wetland habitat in the Golden Gate Estates, and providing additional recharge to the County wellfield along Everglades Blvd. It appears that the septic tank leach field in multiple homes may be affected by increases in groundwater elevation. The County would have to consider appropriate actions to mitigate the impact on these homes. It may be possible to fit the proposed culverts with operable flap gates or drop structures to minimize impacts to downstream property owners.

The largest concern of the analysis is the excess flows leaving the North Belle Meade area and entering Henderson Creek. The additional flows may increase the wet season flow surplus to Rookery Bay. Additional projects would likely be required to minimize potential impacts to Rookery Bay and the Ten Thousand Island estuaries. The use of a smaller diversion structure would reduce the risk to the Ten Thousand Islands Estuary, but would decrease the benefit to Naples Bay.

1. Task 1- Enhance Database Developed in Phase I

1.1. Introduction

The goals of the analyses summarized in this technical memorandum are:

- To review basin information used in the modeling effort,
- To review and update wetland delineations, and
- To summarize the results of wetland functional assessment based on data collected in previous studies.

This information will provide an initial baseline to guide restoration activities in the study area.

1.2. Objectives

This technical memorandum addresses Task 1: Enhance Database Developed in Phase 1. This task includes three subtasks.

- Sub-task 1 was to extract a digital elevation model (DEM) for the study area based on the 2008 LiDAR data. This dataset was used to delineate basins and sub-basins within the project area. The defined basins and sub-basins were then compared to existing basin maps and the results were used later in the project to verify that the hydrologic/hydraulic model that was developed for the study area routes flows correctly.
- Sub-task 2 was to review the wetland delineations defined in the Horsepen Strand Conservation Area Phase 1 report and to update the shapefile based on the new DEM and updated land use data. Limited ground-truthing was completed to support this sub-task and sub-task 3. This information, combined with the results of sub-task 3 was used to help determine the location of potential wetland restoration activities.
- Sub-task 3 was to complete an initial wetland evaluation based on review of available data. The results of the functional assessment were used to provide initial guidance on the types of restoration activities needed in different parts of the project area.

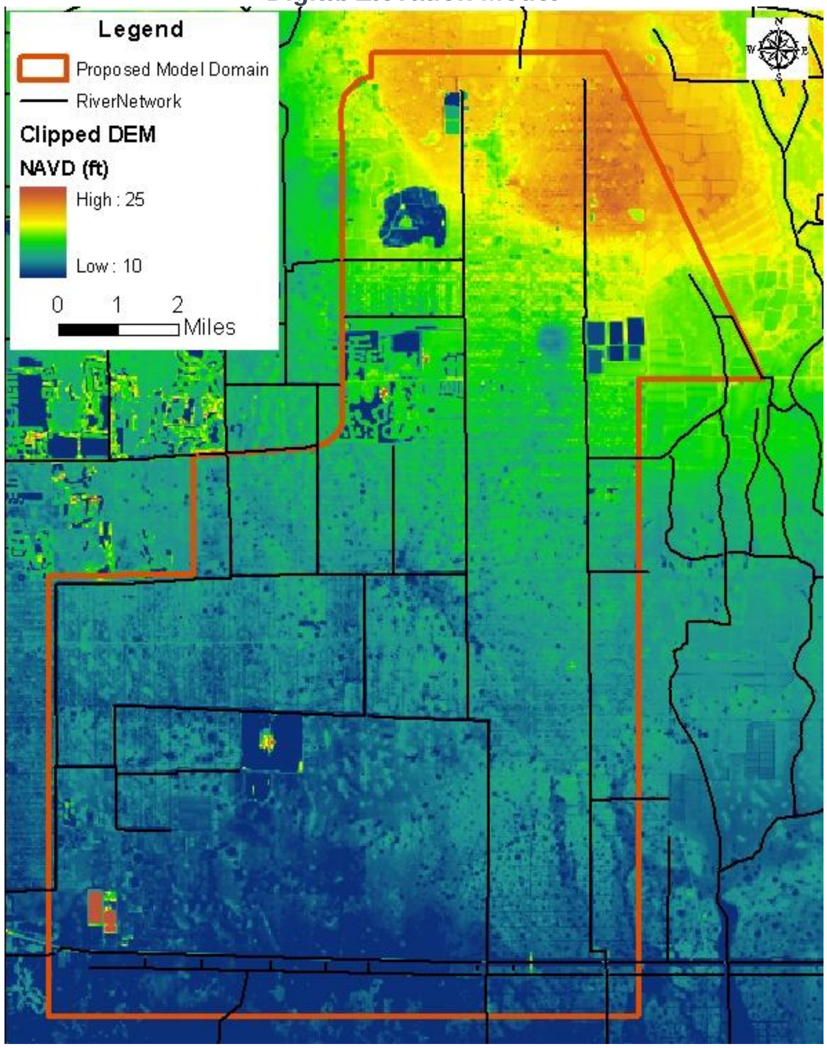
The methods and results for each sub-task are described below.

1.3. Sub-Task 1 – Basin and Sub-basin Delineation

This section describes the methods used to delineate the sub-basins and basins derived from the 2008 DEM. The results were compared to previously defined basin boundaries to ensure that the model accurately reflects flow patterns in the study area.

A DEM for the study area was extracted from the county-wide DEM developed from the 2008 LiDAR survey. The data was required to meet or exceed a 3.8-foot horizontal accuracy and 0.6-foot vertical accuracy (Woolpert, 2009). The coverage area of the clipped DEM is shown in **Figure 1**.

Figure 1. North Golden Gate Estates Flowway Restoration Project Digital Elevation Model



Automated ArcHydro tools were applied to the study area DEM to define sub-basins. The first step of the automated process is to identify sink points. This was performed using the ArcHydro Sink Selection routine and/or manual identification of storage areas based upon terrain and wetland features. Wall features were also used to define sub-basins to canal networks and to match internal Environmental Resource Permit (ERP) drainage boundaries. Using these additional layers, a hydraulically correct DEM was created using the “ArcHydro Fill with sinks” tool. Finally, polygon features were created from the hydraulically correct DEM. The polygon features were reviewed for reasonableness against aerial photography and DEM ridgelines and modified as appropriate to define the sub-basins in the study area. The sub-basins define the localized collection areas based on existing features. The sub-basins were aggregated to define the larger basins at a scale comparable to the existing Southwest Florida Feasibility Study (SWFFS) basin boundaries. Several of the defined basins were split based on the presence of major roads such as Oil Well Rd.

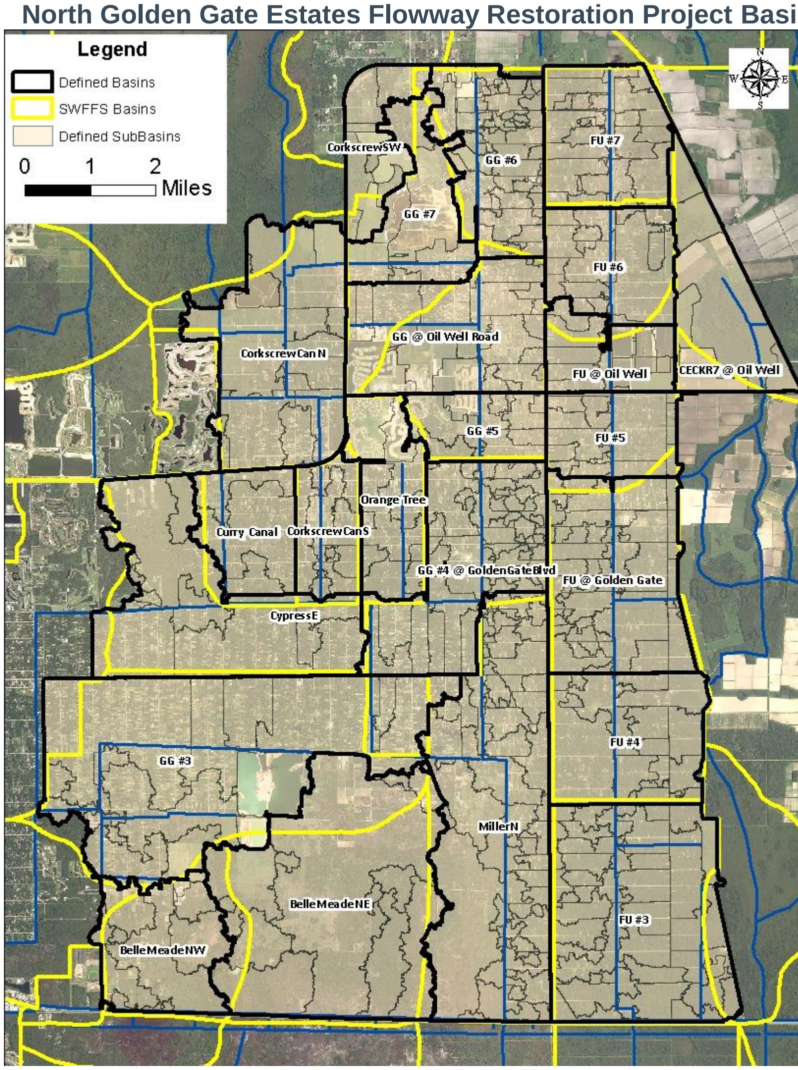
These results were compared to the hydrologic basin boundaries defined by the South Florida Water Management District (SFWMD) for the SWFFS. Those basins have been used to provide consistent boundaries for modeling and analysis in southwest Florida. The sub-basins and basin delineated in this task are shown in **Figure 2.** The figure also shows the existing SWFFS basin delineations as available from the SFWMD (DBHYDRO).

The comparative results shown in **Figure 2** suggest that two of the SWFFS basins in this area may be incorrect. The first is basin GG #7. The results of this sub-task indicate that the southern portion of the previously defined basin is intercepted by a local canal and enters the Golden Gate canal network south of the GG7 structure. This difference may have an effect on the timing of flow to the Golden Gate Canal, but is unlikely to affect the total volume of runoff to the canal.

The second basin is the BelleMeade NE Basin. The defined basin suggests that the urbanized area south of the Golden Gate Main and C-1 Connector canals and east of the mining operation should be included as part of the drainage area flowing to the south. This is inconsistent with the SWFFS basin. It is possible that local drainage features that are not captured in the DEM, do in fact direct flows to the north. Anecdotal (Tim Nance, 2010) information also indicates that large volumes of water may not flow to the south. The information suggests that any overland flow to the south from the urbanized area is restricted by a small east-west road along the SWFFS basin boundary and causes localized flooding. This inconsistency was evaluated during model development.

Overall, the basins defined during this analysis are very similar to the SWFFS basins suggesting that the use of the SWFFS basins to support modeling efforts is generally appropriate with the changes identified in this analysis.

Figure 2. North Golden Gate Estates Flowway Restoration Project Basin Delineation



1.4. Sub-Task 2 - Wetland Delineation

As stated previously, this sub-task, combined with the results of sub-task 3 were used to help determine the location of potential wetland restoration activities in the study area. In this sub-task task, the 2008 land use map available from the SFWMD was modified to reflect the presence of natural areas between the Golden Gate Main/Miller Canals and the Faka Union Canal. The land use map included many polygons that were classified as both rural residential and as a natural area (typically mesic or hydric flatwood).

In order to accurately reflect the actual land use, these polygons were examined and divided as needed to distinguish between the residential areas and the natural areas. The modified land use file was used as the basis of the updates to the wetland delineation maps and was used as a model input file.

Subsequently, the wetland shapefiles generated during Phase 1 of the Horsepen Strand Restoration Project (HSRP) (BRA, 2008) and data from the National Wetland Inventory (NWI) (USFWS, 2010) were overlaid on the modified 2008 land use map. The wetland maps were also compared against the topographical data and aerial photography. The comparison focused on the lands between the Golden Gate/Miller Canal on the west and the Faka Union Canal on the east. The evaluation also considered those lands in the North Belle Meade area. The wetland delineation maps generated by the Horsepen Strand Restoration Project did not consider all lands between the canal systems.

The comparative analysis identified several inconsistencies between the wetlands identified in the modified 2008 land coverage shapefile and previously defined wetland maps. In general, these inconsistencies included:

- Areas identified as wetlands in the 2008 land coverage shapefile that were outside of the wetland boundaries as defined by the HSRP or the NWI.
- Areas defined as uplands (mesic flatwood) in the 2008 land coverage shapefile that were identified as wetlands by the HSRP or the NWI.
- Potential wetland areas that are characterized by low topographical relief that were not defined as wetland areas in either the modified 2008 land coverage or in the wetland maps defined by the HSRP or the NWI. These areas included lands dredged to provide material for residential development.

Limited ground-truthing was conducted to evaluate inconsistencies in the study area and the land use map was further modified based on the visual survey. The objective was to determine the appropriate land cover classification in the areas with inconsistent information. The results of the wetland delineation evaluation are shown in **Figure 3**.

The results shown in **Figure 3** indicate that the wetland delineations are very similar in the areas where the HSR and Northern Golden Gate Estates Flowway Restoration Project (NGGEFRP) boundaries intersect**. Figure 4** provides a closer view of the northern portion of the study area. The primary differences in the wetland delineations are north of the Panther Walk wetland system and west of the Winchester Head wetland. In these areas, the limited ground-truthing confirmed that these areas are hydric flatwood rather than mesic uplands. **Figure 4** also shows the location of many small ponds and marsh areas that were created by dredging activities during construction of homes in the study area.

The nano-wetlands associated with individual lots may have many benefits depending on the location and bottom elevation relative to the seasonal high water level. If wet season storage is available in these nano-wetlands, the County should consider policies that would require homeowners to direct runoff from yards and impervious surfaces toward the local wetland area. This would reduce discharge to the roadside swales and provide additional water quality treatment.

Figure 3. NGGE Flowway Restoration Project

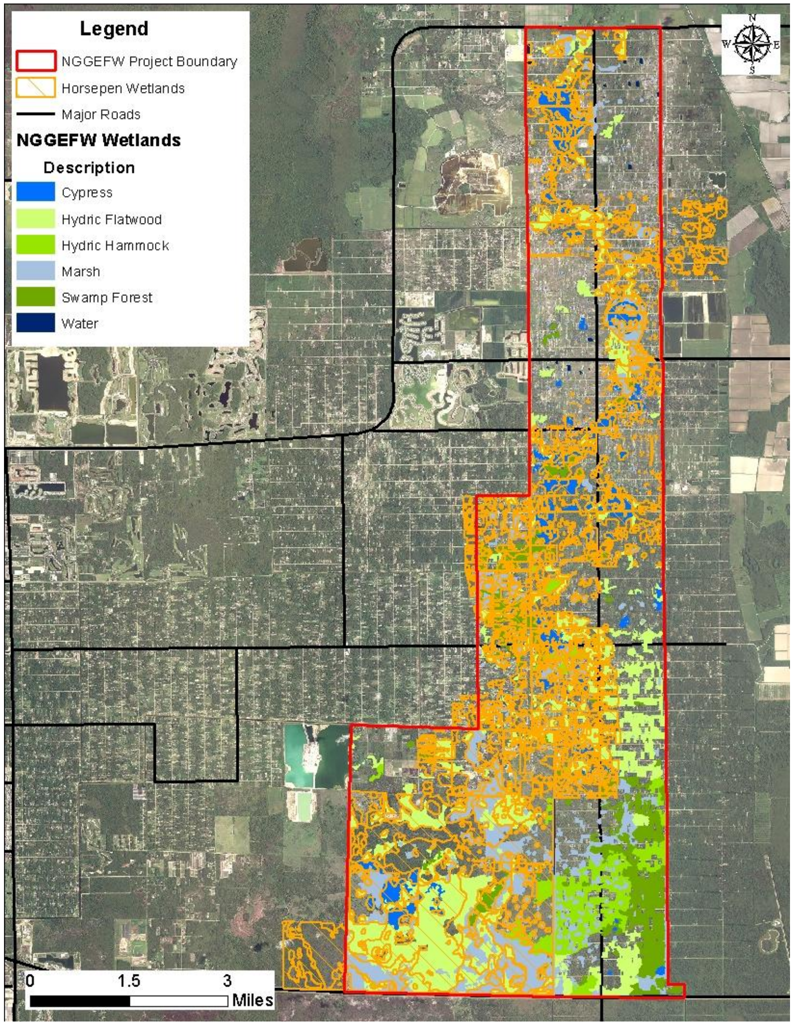
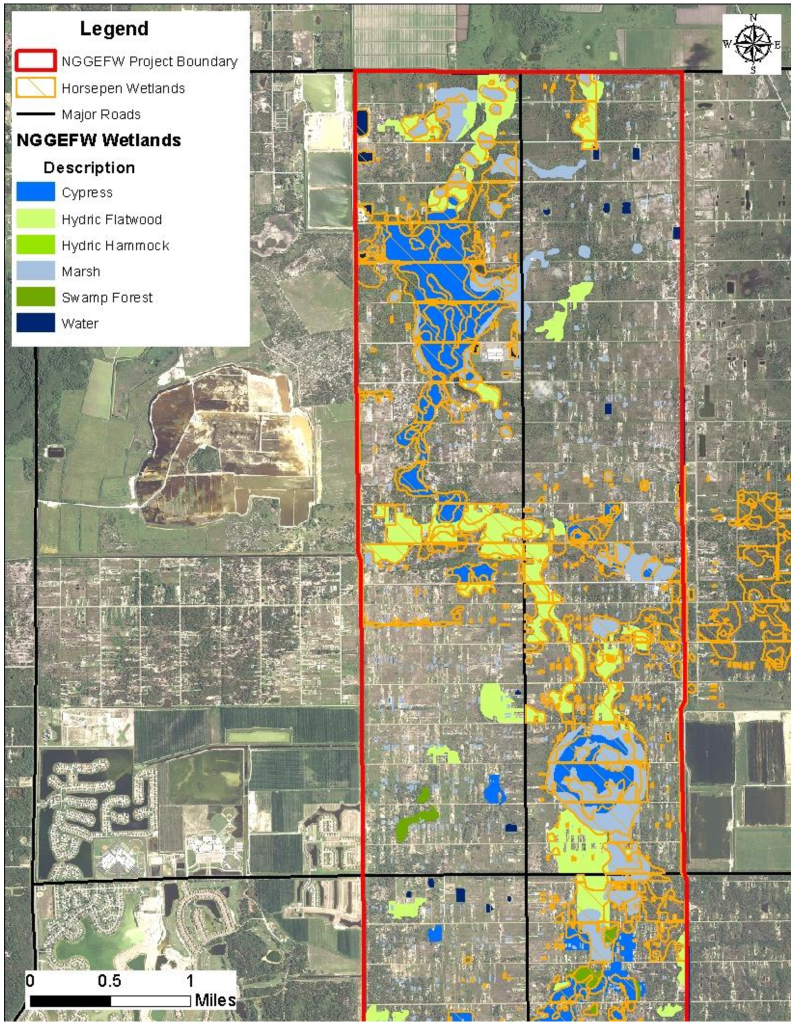


Figure 4. HSRP Wetland Delineation vs. 2008 Wetland Delineation



1.5. Sub-Task 3 - Evaluation of Wetland Function

The objective of this work as to provide initial guidance on the location and types of restoration activities that were evaluated in future tasks. For this sub-task, data from the Collier County Watershed Management Plan (CCWMP) project and the HSRP were reviewed to provide an initial functional assessment of the wetlands in the project area. The following paragraphs describe land use changes in the study area from the pre-development period and document the reported functional level of the remaining wetland systems.

Land Use Changes. In the pre-development condition (**Table 1**), the NGGEFRP area (28,138 acres) was composed of approximately 17,715 acres of freshwater wetlands (63 percent) and approximately 10,423 acres of undeveloped uplands (37 percent). By 2008, over 5,464 acres (19 percent) of the study area had been converted to urban-related development. Approximately 6,563 acres (37 percent) of freshwater wetlands (freshwater marsh, cypress, hydric flatwoods, swamp forest, wet prairie, hydric hammock) had been lost to development or due to changes in hydrology. **Figure 5** depicts the net changes due to development between the SWFFS Pre-Development Vegetation Model (PDVM) and the modified 2008 land cover.

Table 1 shows that the wetland communities with the greatest losses were to cypress and hydric flatwoods. The losses can be attributed to urban growth and to changes in hydrology associated with construction of the Golden Gate drainage canals. The data shows a loss of hydric flatwood that exceeds 5,000 acres and a loss of slightly more than 2,800 acres of cypress. However, more than 3,200 acres of freshwater marsh are present in 2008 that were not present in the predevelopment condition. Many of the freshwater marshes and ponds in the NGGEFRP area created during construction of single family residences. The construction process included scraping and/or excavation for fill material on individual lots. The fill material was (and is) used to construct elevated septic leach fields and home foundations. Many of the excavated areas are now characterized as freshwater marshes or ponds.

Table 1 also indicates that there a slight loss in total upland (mesic) communities. However, **Table 2** shows that more than 2,200 acres of predevelopment upland lands have been converted to urban land uses. The increase in mesic communities is linked to changes in the hydrology of predevelopment wetland areas that are now characterized by shorter hydroperiods and less water storage.

Evaluation of Wetland Function: The Horsepen Strand Conservation Area Phase I report (BRA, 2008) states that wetland degradation is due to ditching and piping resulting from development of the Northern Golden Gate Estates. The development "consisted of a series of east-west paved roadways with open ditches on either side at approximately quarter mile intervals." The report also stated that "Water that historically was stored in a wetland flowway is now diverted via open ditches to the Golden Gate, Miller, and Faka Union Canals."

Figure 5. North Golden Gate Estates Flowway Restoration Project Land Use and Land Cover Changes from Pre-Development vs. 2008 (GIS Source Data from SFWMD)

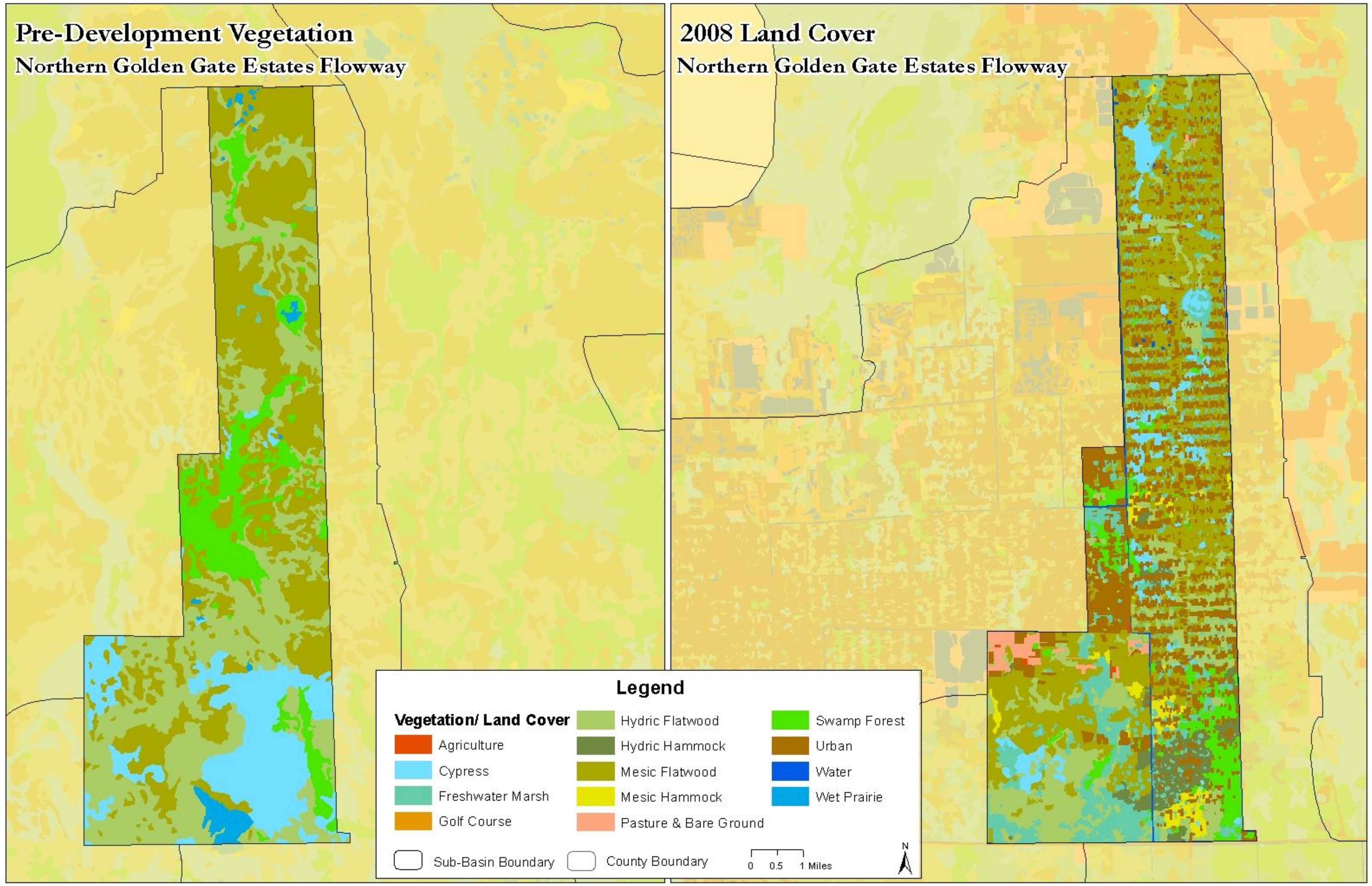


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Table 1. North Golden Gate Estates Flowway Restoration Project Land Use and Land Cover Changes from Pre-Development vs. 2008

Northern Golden Gate Estates Flowway Project Area				
Vegetation Type	Pre-Development Land Cover		2008 Land Cover	
	Acres	Percent of Total	Acres	Percent of Total
Agriculture	-	-	40	0
Freshwater Marsh	70	0	3,346	12
Urban	-	-	5,013	18
Cypress	4,224	15	1,393	5
Pasture & Bare Ground	-	-	411	1
Hydric Flatwood	9,835	35	4,385	16
Swamp Forest	3,052	11	1,014	4
Mesic Flatwood	10,423	37	10,072	36
Water	-	-	245	1
Golf Course	-	-	-	-
Mesic Hammock	-	-	613	2
Wet Prairie	533	2	-	-
Mangrove	-	-	-	-
Hydric Hammock	-	-	1,014	4
Tidal Marsh	-	-	-	-
Xeric Hammock	-	-	-	-
Beach	-	-	-	-
Xeric Flatwood	-	-	-	-
Total	28,138	100	28,138	100

Table 2. North Golden Gate Estates Project Area Conversions from Pre-Development to 2008 (Acres)

Pre-Development Land Cover	Agriculture	Golf Course	Pasture & Bare Ground	Urban
Freshwater Marsh	0	0	0	12
Cypress	27	0	190	519
Hydric Flatwood	12	0	151	1,589
Swamp Forest	0	0	0	587
Mesic Flatwood	0	0	70	2,282
Wet Prairie	0	0	0	24

A comparison of the hydrological characteristics of pre-development and 2007 vegetation communities was completed for the CCWMP and is described in detail in Section 2.8 of Volume 4 (Atkins, 2011). The two findings that are most relevant to this project are areas of potential storage capacity and the "scoring" the hydrologic function of land uses in the study area.

Figure 6 shows the areas identified for potential additional wet season water storage in the NGGEFWR project area. Overall, approximately 1,800 acres of undeveloped lands (including over 1,200 acres in the Faka Union watershed portion of the study area) have capacity for additional wet season storage of at least 0.5 feet up to 2.0 feet. The largest opportunity for storage as defined by the CCWMP, and based strictly on the difference in hydrological characteristics between pre-development and 2007 vegetation communities, is located in the north Belle Meade area located north of I-75, south of the Golden Gate Main Canal and west of Miller Canal. The CCWMP concluded that restoration of hydrology in this area could lead to large-scale improvements in both functional value and hydrological storage.

The CCWMP also used hydrology scores to characterize the effects of depth and duration (hydroperiod) of inundation. The hydrologic scoring method (Volume 4) developed for the CCWMP assigns values by comparing modeled hydrology results against the expected hydrology of the predevelopment vegetative conditions.

The hydrology scoring in the CCWMP represents the functional value of a parcel of land based on the degree to which the parcel retains the same hydrological characteristics as its pre-development reference condition. Pre-development hydrological conditions were estimated based on the typical range of depth and duration (hydroperiod) of inundation of the vegetation community present in the PDVM per **Table 3** (Mike Duever, personal communication). The predicted average depth and hydroperiod were determined from the Collier County Existing Conditions Model (Atkins, 2011) developed for the CCWMP.

Figure 6. North Golden Gate Estates Flowway Restoration Project Areas of Potential Additional Storage

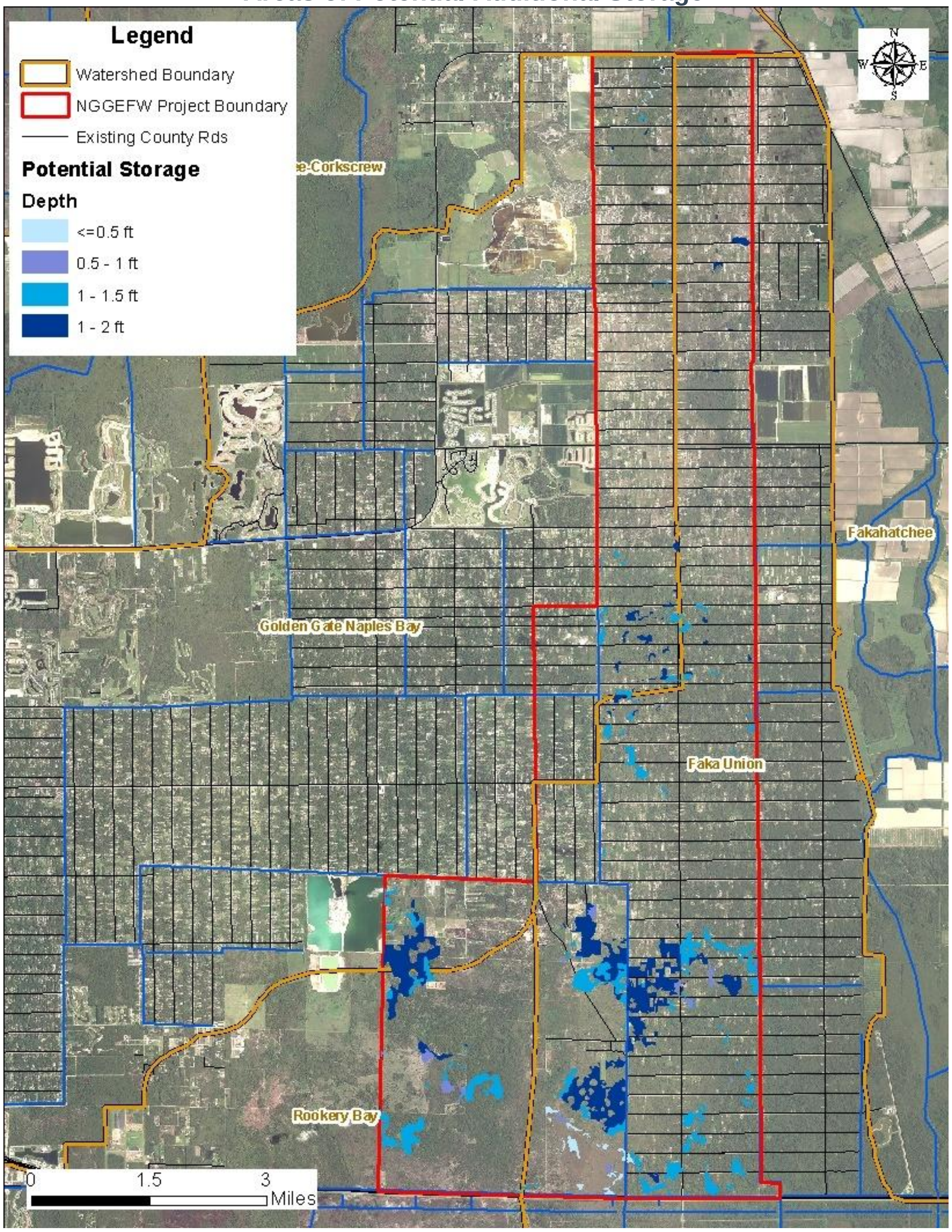


Table 3. Hydrologic Regimes of Major Southwest Florida Plant Communities

Plant Community	Duration (months)	Seasonal Water Depth (inches)	
		Wet	Dry (1,10)*
Xeric Flatwood Xeric Hammock	0	≤-24	-60, -90
Mesic Flatwood Mesic Hammock	≤1	≤2	-46, -76
Hydric Flatwood Hydric Hammock	1–2	2–6	-30, -60
Wet Prairie Dwarf Cypress	2–6	6–12	-24, -54
Freshwater Marsh	6–10	12–24	-6, -46
Cypress	6–8	12–18	-16, -46
Swamp Forest	8–10	18–24	-6, -36
Open Water	>10	≥24	< 24, -6
Tidal Marsh Mangrove Beach	Tidal	Tidal	Tidal

* 1 = average year low water; 10 = 1 in 10 year drought, July 2002

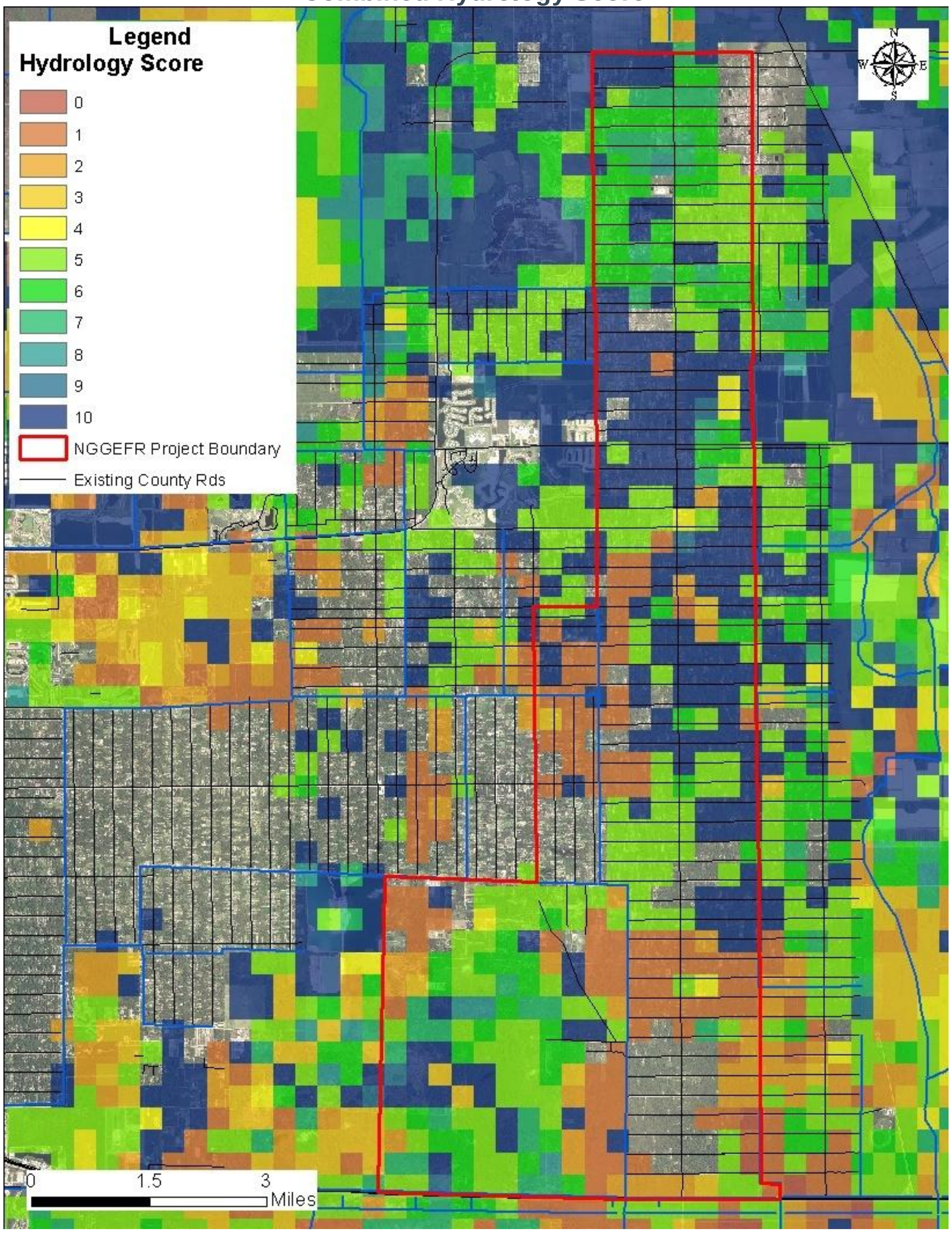
Areas for which model predicted hydrological conditions were in the normal range of the pre-development conditions were designated with higher scores, while areas that did not meet (i.e. shorter duration hydroperiod or less depth of inundation) predevelopment conditions were assigned lower scores.

For example:

- No change from pre-development would result in a score of 10.
- Total loss of hydrology (e.g., a cell dominated by a pre-development wetland or open water body but which now experiences no inundation) would result in a score of zero (0).

Figure 1.7 shows the calculated combined hydrology score from the Collier County Existing Conditions MIKE SHE/MIKE 11 model within the NGGEFRP area. The cells with the lowest combined hydrology scores are consistent with the areas where there is the greatest capacity to store additional water (**Figure 6**). In other areas, cells with scores in the 5 – 7 range indicate that hydrologic improvement will result from extending the hydroperiod over much of the project area without significantly changing the depth of water stored.

Figure 7. North Golden Gate Estates Flowway Restoration Project Combined Hydrology Score



1.6. Conclusions

The following conclusions were reached during this task:

- The SWFFS basins and the basins defined in the task are similar indicating that the basins are generally appropriate for modeling purposes. The results indicate that two basins may require modification; however, the differences are unlike to affect the overall hydrology of the study area. These potential differences were considered during the modeling tasks.
- The wetland maps developed for the HSRP and the NWI are not consistent and omit wetland areas created by grading and dredging activities that support residential development. Many isolated freshwater marshes and ponds were created as a result of these dredging activities. The revised wetland maps were used to guide restoration activities and to better define land uses in the modeling effort
- More than 8,200 acres of hydric flatwood and cypress wetlands have been lost in the study area. These areas have been converted to urban land uses, or the hydrology has changed such that the land cover is now consistent with upland mesic communities. This information was used to provide initial guidance on potential restoration activities that will store water at a greater depth, or will extend the current hydroperiod to better match redevelopment levels

2. Task 2 - Model Development and Calibration

2.1. Introduction

This document summarizes sub-tasks 2-1 through 2-3 of the Scope of Work (SOW). The model was used to establish baseline conditions and served as the basis of comparison for alternative design strategies.

2.2. Objectives

This technical memorandum addresses the Model Development portions of Task 2 of the SOW. This memorandum describes the work completed for three subtasks.

- Sub-task 2.1 is to develop a local scale model from the regional Collier County Existing Conditions Model (CC ECM). The task also specified that boundary conditions were extracted from the CC ECM for use in the local scale model.
- Sub-task 2.2 identified specific items to be included in the local scale model. These include additional hydraulic features based on field work and review of Environmental Resource Permits (ERP), as well as modified land use data.
- Sub-task 2.3 included calibration of the local scale model using results from the simulation period of January 1, 2002 to October 31, 2007. Calibration was accomplished by comparing model results to observation data.

Subtasks 2.1 and 2.2 are discussed simultaneously in Sections 2.3 – 2.10 since the two are inter-related. The calibration process is presented in Section 2.11.

2.3. Development of a Local Scale MIKE SHE/MIKE 11 Model

This section describes the steps and methods used to develop the local scale MIKE SHE/MIKE 11 from the Collier County model. A revised version of the CC ECM was used as the basis of the local scale model. The local scale model was extracted from the MIKE SHE/MIKE 11 model identified as "CC_EC_Calibrated_rev3a.SHE". This model varies from the original CC ECM (CC_EC_Calibrated_rev1.SHE) in that it includes a 7-layer groundwater component, hourly rainfall and evaporation input based on the NEXRAD grid, and additional logic for many of the operable structures in the model domain. A complete summary of the model revisions are found in the Enhanced Model Development Technical Memorandum (DHI, 2011).

The following steps were completed to develop the local scale model for the North Golden Gate Estates Flowway Restoration Project (NGGEFRP) area.

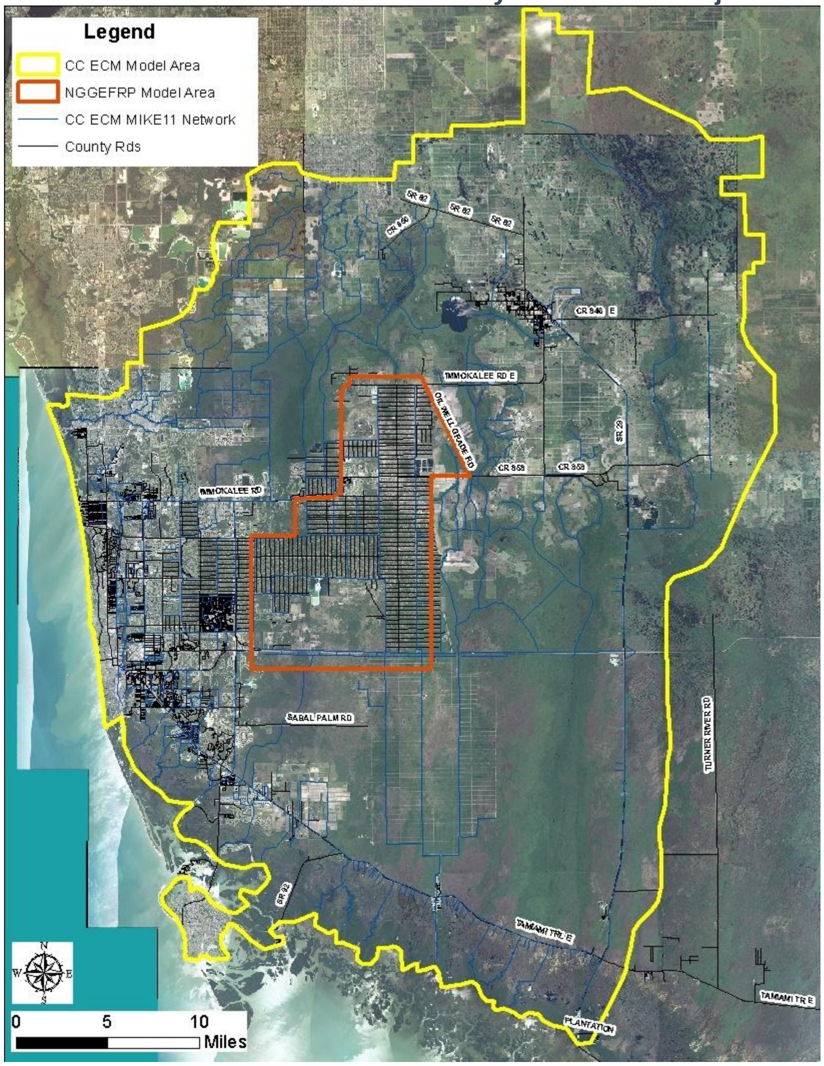
- Define the local scale model domain and grid size;
- Define land use input data for the local scale model;
- Define topography for the local scale model;
- Reduce the MIKE 11 River network to correspond to the new domain;
- Extract boundary conditions from the CC ECM; and
- Run initial simulation and compare results against the CC ECM
- Add additional hydraulic features

Each step is discussed below.

2.4. Define the local scale model domain

The domain for the local scale model of the North Golden Gate Estates Flowway Restoration Project (NGGEFRP) is shown in **Figure 8**. The local scale model will utilize a 500-foot grid instead of the 1500-foot grid used in the CC ECM.

Figure 8. North Golden Gate Estates Flowway Restoration Project Model Domain



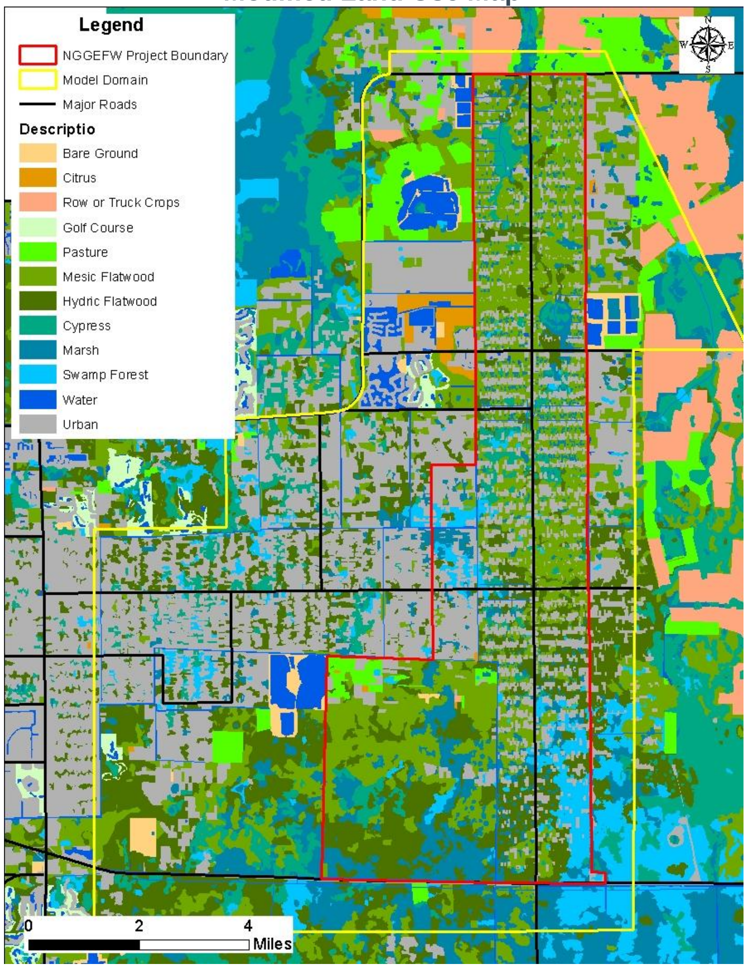
2.5. Define land use input data for the local scale model

This section describes new or modified data applied to the local scale model.

Land Use Data: The NGGEFRP local scale model land use input data is based upon the 2008 land use shapefile available from the South Florida Water Management District (SFWMD). In this polygon shapefile, many polygons have a land use description of “rural residential” and a land cover description consistent with natural areas such as hydric flatwood or mesic flatwood. In the regional CC ECM, these polygons were categorized as “Urban Low Density” (ULD)

In the local scale model, it was not appropriate to generally classify the rural residential land use polygons as ULD; therefore, in the area between the Faka Union Canal on the east, and the Golden Gate and Miller Canals on the west, the land use polygons were edited such that the area of the actual residences was described as urban low density and the natural areas were classified based on the predominant vegetative cover. This provided a better representation of the actual distribution of land uses. The modified land use distribution is shown in **Figure 9.**

Figure 9. North Golden Gate Estates Flowway Restoration Project Modified Land Use Map



In the model, a code was assigned to each cell that equaled the predominant land use within the cell. Other model inputs, such as the paved runoff coefficient, the overland roughness coefficient, on-site storage, etc. were assigned based on the land use code. The values used in the model are shown in **Table 4**.

Table 4. Local Scale MIKE SHE/MIKE 11 Model Land Use Based Input Parameters

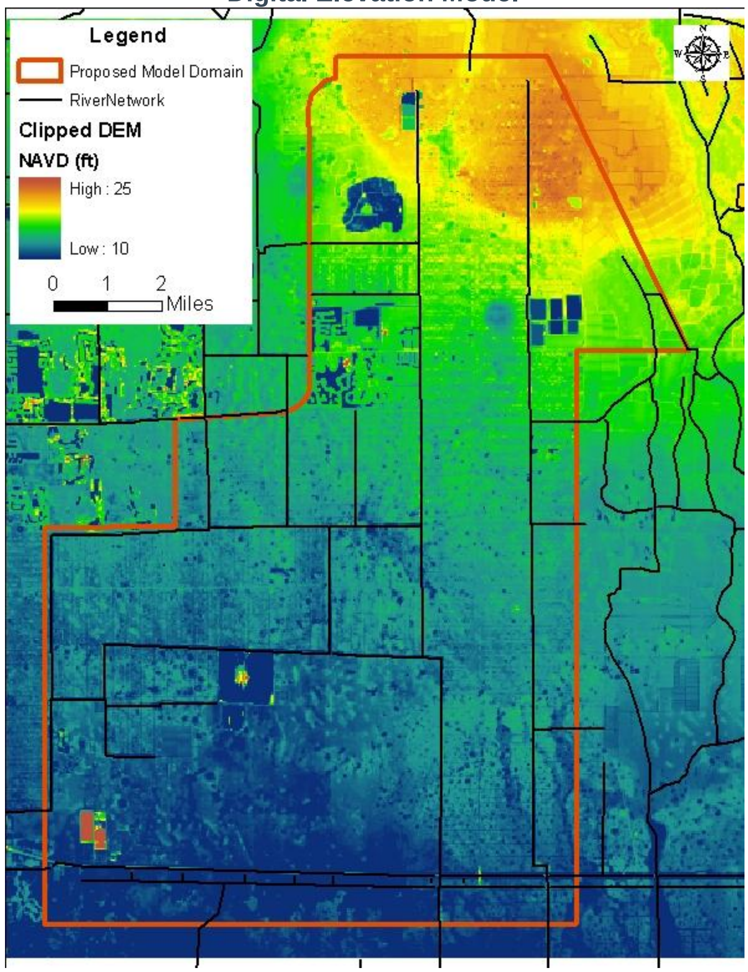
MSHE Code	Land Use or Vegetation Type	Overland Manning's (n)	Detention Storage (inches)	Paved Runoff Fraction	Drainage Depth (ft)	Drainage Time Constant (1/day)
1	Citrus	0.17	1	0	0.5	0.25
2	Pasture	0.14	1.2	0	0.5	0.25
3	Sugar Cane	0.17	1	0	0.5	0.25
5	Truck Crops	0.17	1	0	0.5	0.25
6	Golf Course	0.14	1.2	0	1	0.25
7	Bare Ground	0.09	1.2	0	0	0
8	Mesic Flatwood	0.20	1.2	0	0	0
9	Mesic Hammock	0.30	1.2	0	0	0
11	Xeric Hammock	0.20	1.2	0	0	0
12	Hydric Flatwood	0.25	1.2	0	0	0
13	Hydric Hammock	0.40	1.2	0	0	0
14	Wet Prairie	0.30	1.2	0	0	0
16	Marsh	0.43	1.2	0	0	0
17	Cypress	0.30	1.2	0	0	0
18	Swamp Forest	0.40	1.2	0	0	0
19	Mangrove	0.20	1.2	0	0	0
20	Water	0.06	1.2	0	0	0
41	Urban Low Density	0.14	1	0.05	0.5	0.25
42	Urban Medium Density	0.12	0.4	0.15	0.75	0.35
43	Urban High Density	0.11	0.13	0.45	1	0.5

2.6. Define topography for the local scale model domain

A DEM for the study area was extracted from the county-wide DEM developed from the 2008 LiDAR survey. The data was required to meet or exceed a 3.8-foot horizontal accuracy and 0.6-foot vertical accuracy (Woolpert, 2009). The coverage area of the clipped DEM is shown in **Figure 10**. The DEM was converted to a model input file that averaged the elevation over each 500-ft grid cell.

In MIKE SHE, the use of grid cells results in the topography being averaged within each cell. The result is that local topographic features, such a roads and berms, may be lost. In order to represent these features, MIKE SHE uses Separated Overland Flow areas to represent the hydrologic effect of these features. Water is not allowed to move from one Overland Flow area to the next unless a pathway is defined in the MIKE 11 river network.

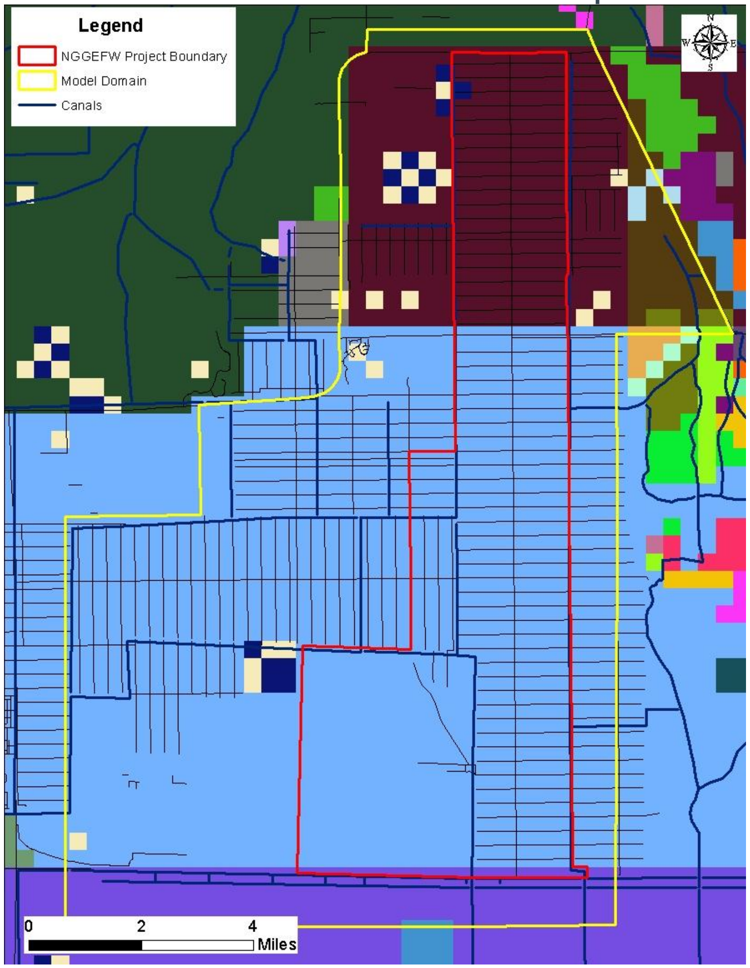
Figure 10. North Golden Gate Estates Flowway Restoration Project Digital Elevation Model



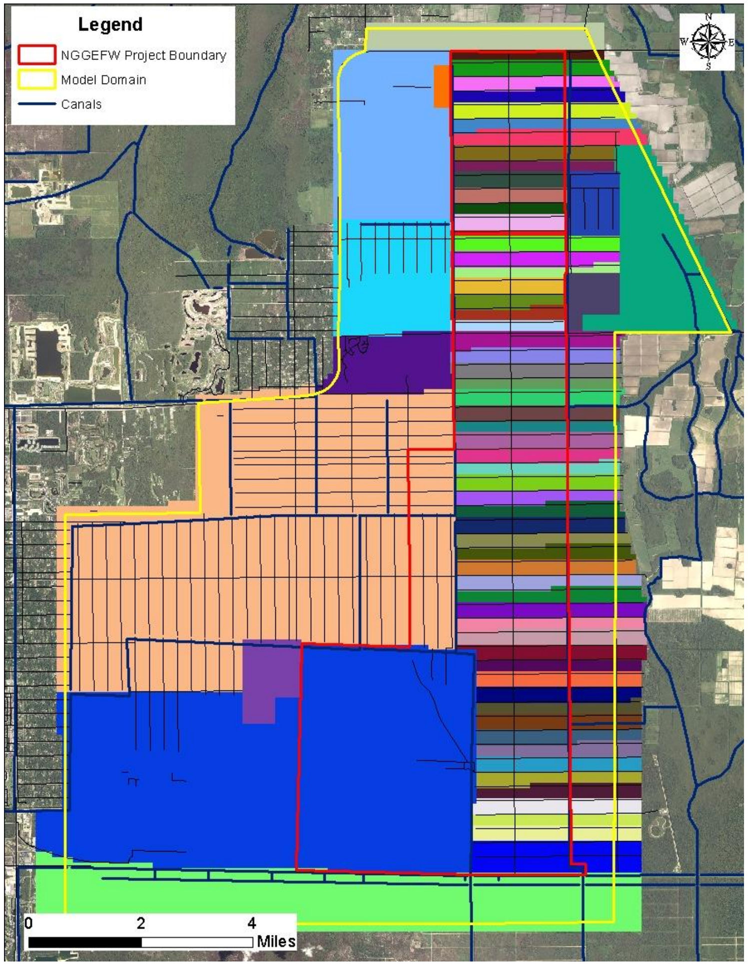
Due to the gird size, the CC ECM assumed that only major roads, such as Oil Well Road, restricted overland flow; therefore, these roads were used to define the boundaries of the Separated Overland Flow areas. The CC ECM assumed that smaller roads did not restrict the movement of water in the overland flow plain.

In the local scale model, the smaller grid size means that there are two or more cells located between each of the cross streets along the Everglades Blvd corridor. The cross-streets and road side swales restrict overland flow from the north to south and direct water to the Golden Gate Canal network. Therefore, Separated Overland Flow areas were defined between each cross street along the Everglades Blvd corridor between the Golden Gate/Miller Canals and the Faka Union Canal. In the model, this will prevent flow from north to south and direct water to the east or west toward the Golden Gate Canal network unless culverts and weirs are defined in the MIKE 11river network. A comparison of the Separated Overland Flow areas is shown in **Figure 11.**

Figure 11. North Golden Gate Estates Flowway Restoration Project Comparison of Separated Overland Flow Areas



Collier County Existing Conditions Model Separated Overland Flow Areas



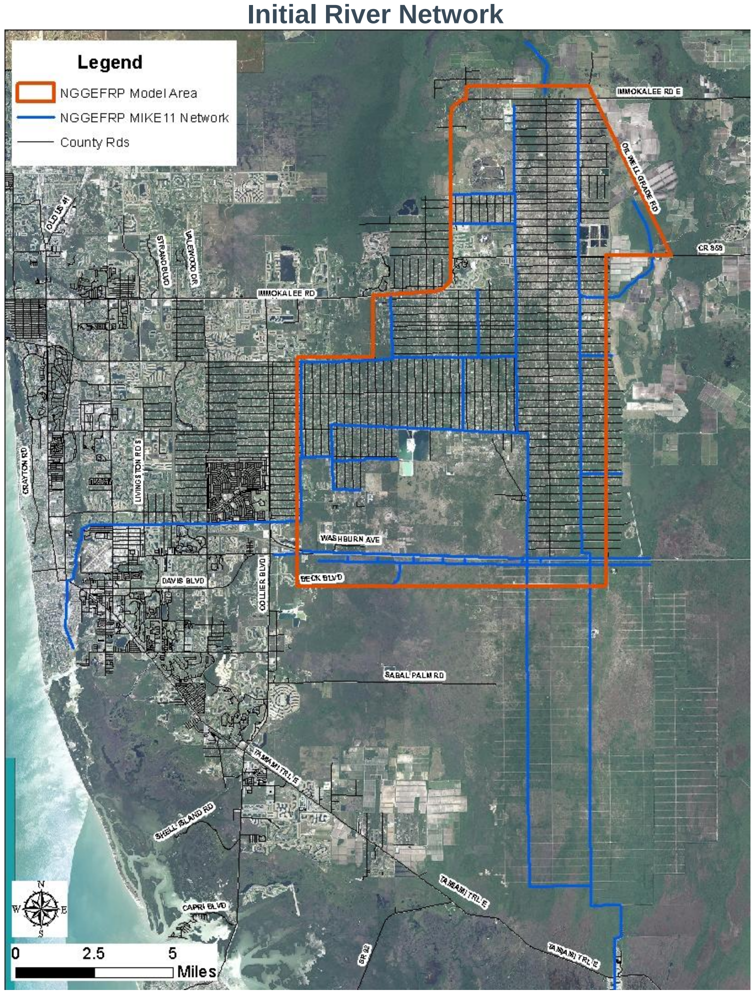
Northern Golden Gate Estates Separated Overland Flow Areas

2.7. Reduce the MIKE 11 River Network

For the NGGEFRP model, the MIKE 11 river network was reduced to the branches that fall within the model domain and the primary canals that discharge to Naples Bay and the Ten Thousand Islands. The reduced MIKE 11 network is shown in **Figure 12**.

In addition to reducing the number of branches in the MIKE 11 network, the coupling to MIKE SHE was also modified and limited to the segments of the branches that are within the NGGEFRP Model Area. MIKE 11 boundary conditions (stage or flow) were extracted from the result files of the CC ECM.

Figure 12. North Golden Gate Estates Flowway Restoration Project Initial River Network



2.8. Extract boundary conditions from the CC ECM

Groundwater Head elevation data was extracted from the CC ECM result files for each of the seven (7) groundwater aquifers to create boundary input files for the local scale model.

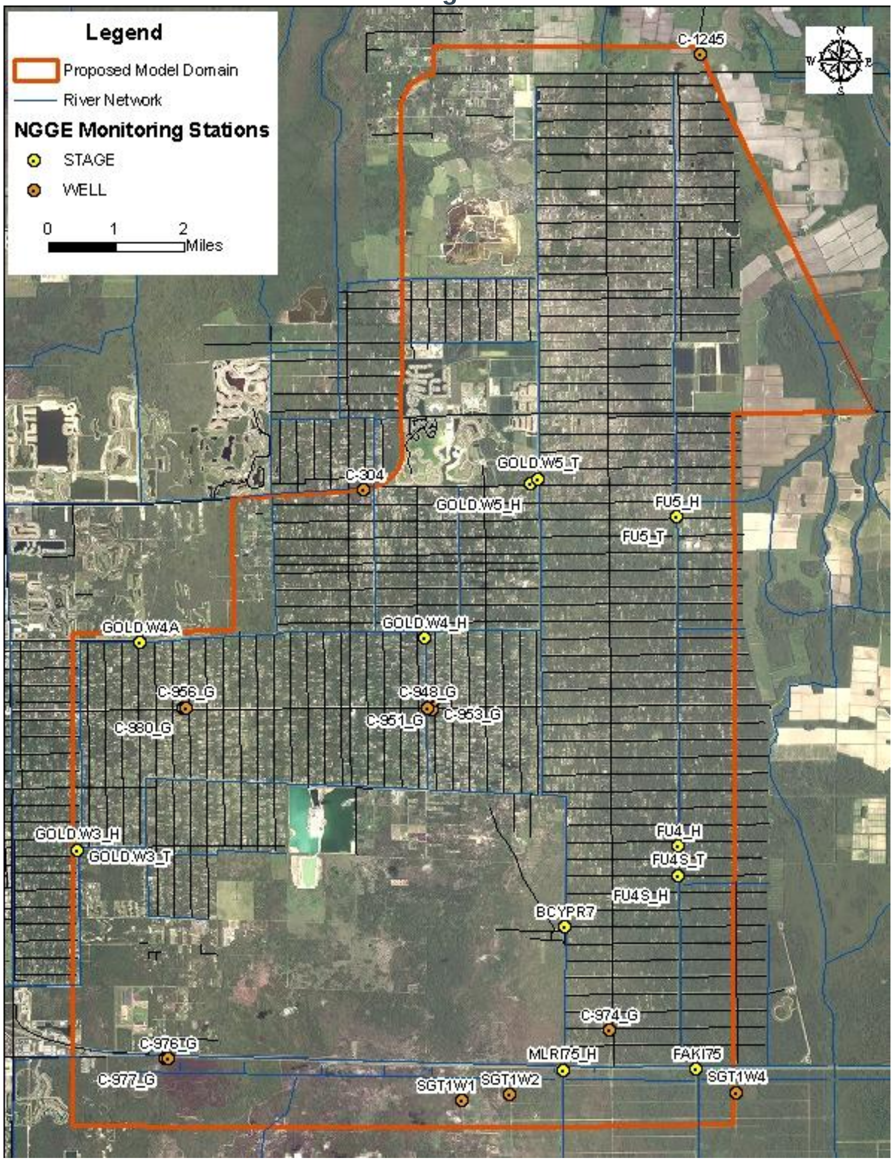
Stage data was also extracted from the MIKE 11 results of the CC ECM to develop boundary condition inputs for the local scale MIKE 11 model.

2.9. Run Initial Simulation and Compare Results to CC ECM

An initial local scale model run (CC_EC_Rev3a_NGGE_Run1.SHE) was completed for the period from January 1, 2002 – October 31, 2007. This initial simulation was completed and compared to results of the CC ECM to verify that the models produce similar results for the calibration period that is defined as November 1, 2002 – October 31, 2007. This initial local scale model included the items described in previous sections.

In order to assess the initial results of the NGGEFRP local scale model, results were compared to those produced in the CC ECM at monitoring stations in the model domain. **Figure 13** shows the location of the observation stations that are located within the local scale model domain.

Figure 13. North Golden Gate Estates Flowway Restoration Project Monitoring Stations

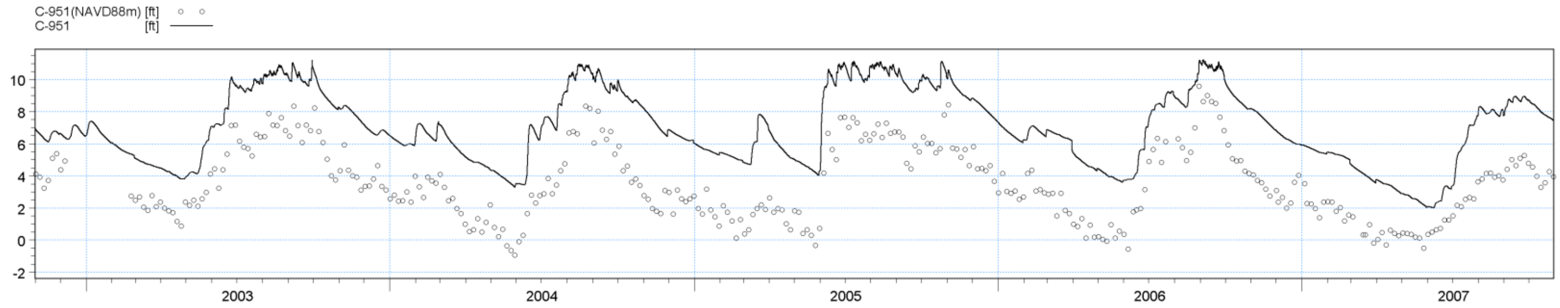


Figures 14 – 18 show comparative results at several of the observation stations. Figures showing the results at all of the monitoring stations that are included in the CC ECM and within the local scale model domain are shown in Appendix A – Comparison of Initial Local Scale Model Results to the CC ECM.

Figure 14. Comparison of Initial Local Scale Model Run to CC ECM Results Monitoring Well C-951

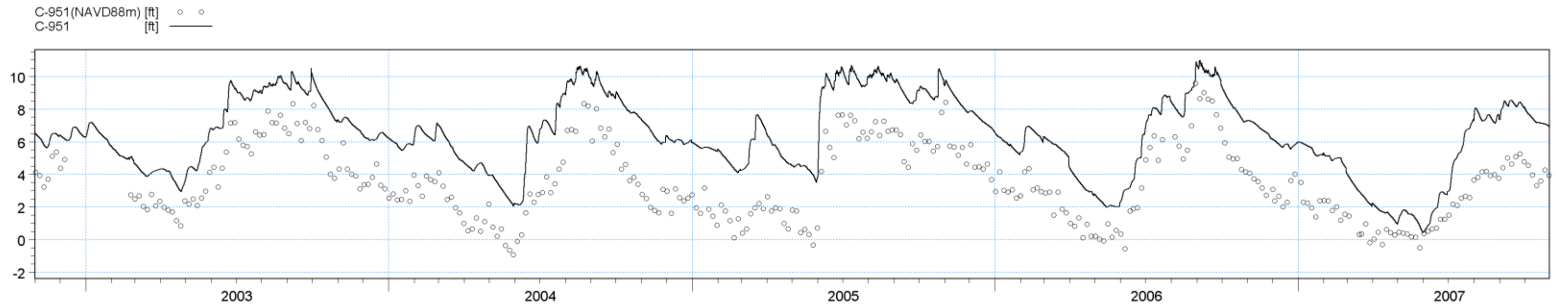
Monitoring Well C-951 (Lower Tamiami Aquifer)

CC ECM Calibration Model



ME=-3.51325
MAE=3.51325
RMSE=3.6148
STDres=0.850808
R(Correlation)=0.931821
R2(Nash_Sutcliffe)=-1.43592

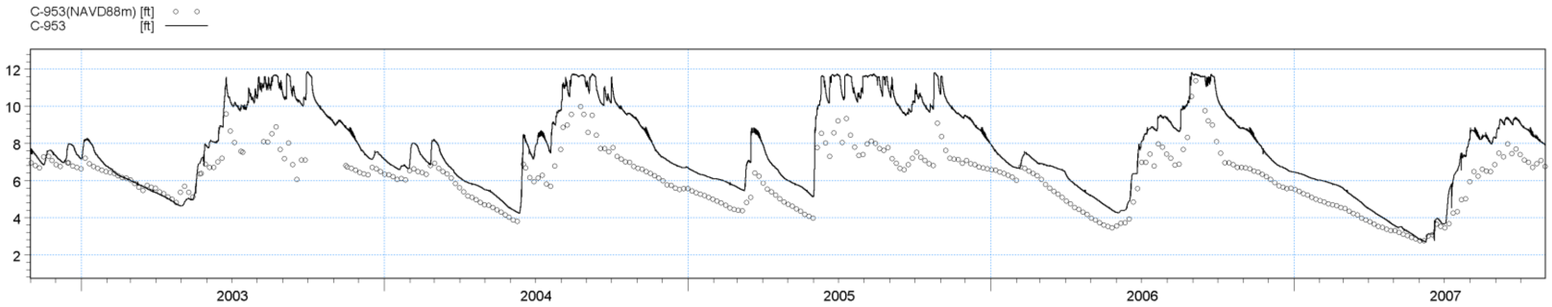
NGGEFRP Local Scale Model



ME=-2.91404
MAE=2.91498
RMSE=3.04706
STDres=0.890493
R(Correlation)=0.928314
R2(Nash_Sutcliffe)=-0.730845

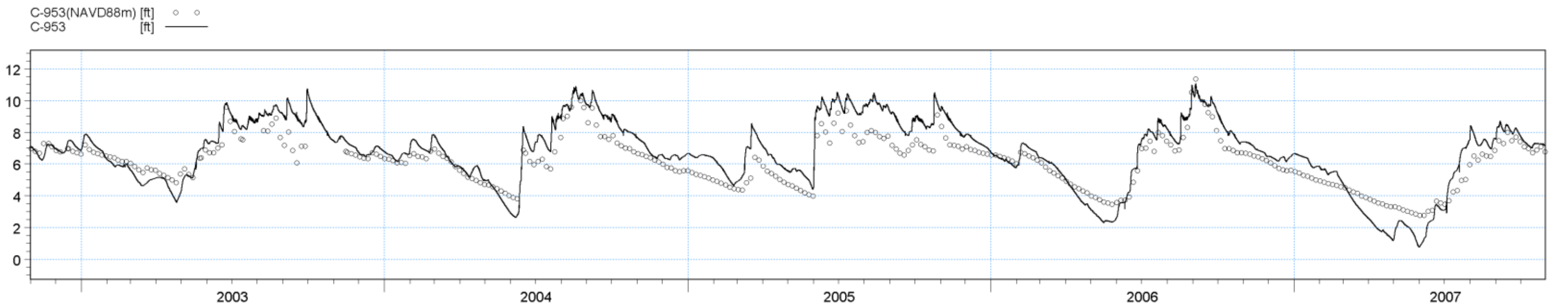
Figure 15. Comparison of Initial Local Scale Model Run to CC ECM Results
Monitoring Well C-953

Monitoring Well C-953 (Water Table Aquifer)
CC ECM Calibration Model



ME=-1.44179
MAE=1.46652
RMSE=1.73764
STDres=0.969861
R(Correlation)=0.92671
R2(Nash_Sutcliffe)=-0.318288

NGGEFRP Local Scale Model

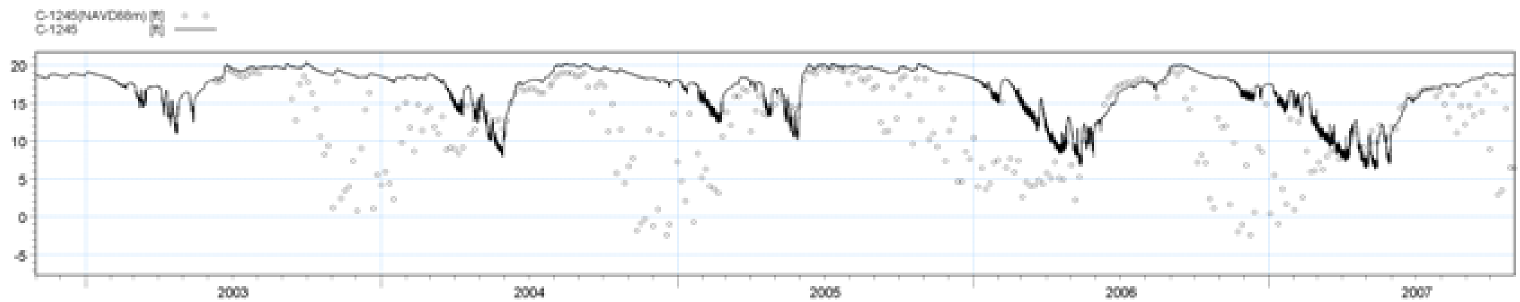


ME=-0.503229
MAE=0.834379
RMSE=0.99198
STDres=0.854859
R(Correlation)=0.93304
R2(Nash_Sutcliffe)=0.570368

Figure 16. Comparison of Initial Local Scale Model Run to CC ECM Results Monitoring Well 1245

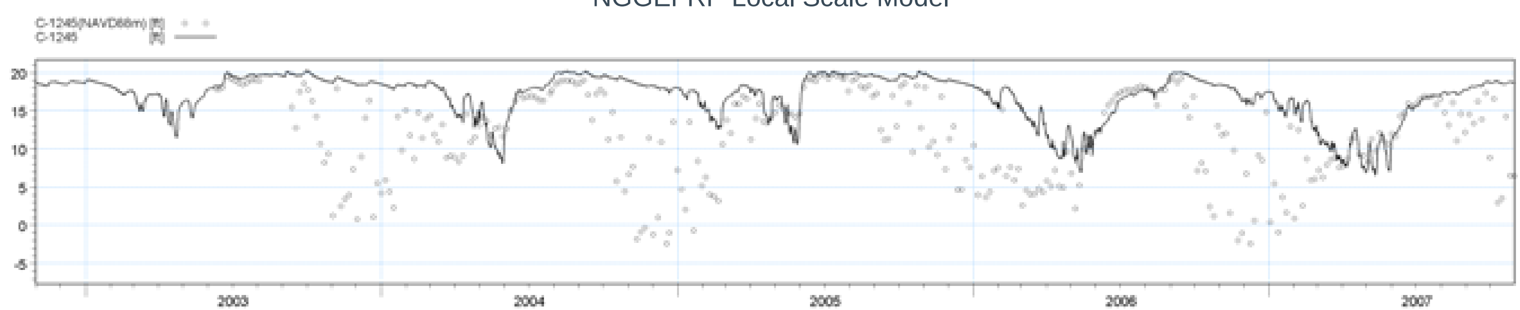
Monitoring Well C-1245 (Lower Tamiami Aquifer)

CC ECM Calibration Model



ME=-5.13074
MAE=5.47871
RMSE=7.62531
STDres=5.64101
R(Correlation)=0.301938
R2(Nash_Sutcliffe)=-0.774761

NGGEFRP Local Scale Model

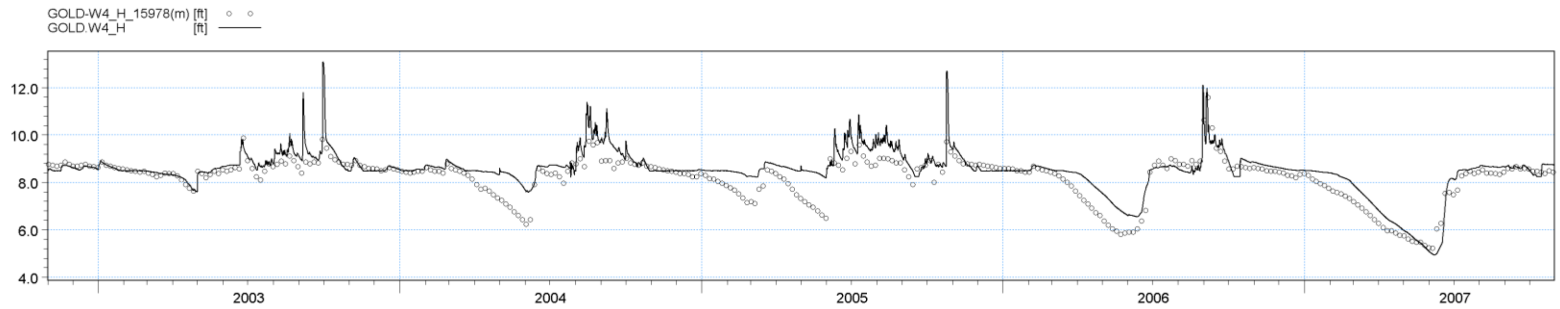


ME=-5.12495
MAE=5.46282
RMSE=7.6115
STDres=5.6276
R(Correlation)=0.303689
R2(Nash_Sutcliffe)=-0.768337

Figure 17. Comparison of Initial Local Scale Model Run to CC ECM Results Stage Monitoring Station GOLD.W4_H

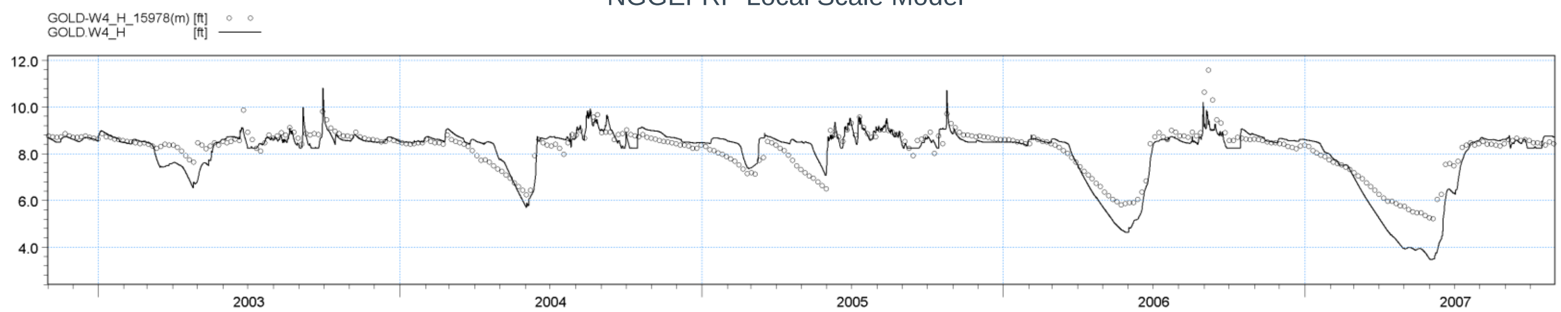
Station GOLD.W4_H (Golden Gate Main Canal)

CC ECM Calibration Model



ME=-0.306972
MAE=0.394381
RMSE=0.548976
STDres=0.455129
R(Correlation)=0.87095
R2(Nash_Sutcliffe)=0.644037

NGGEFRP Local Scale Model

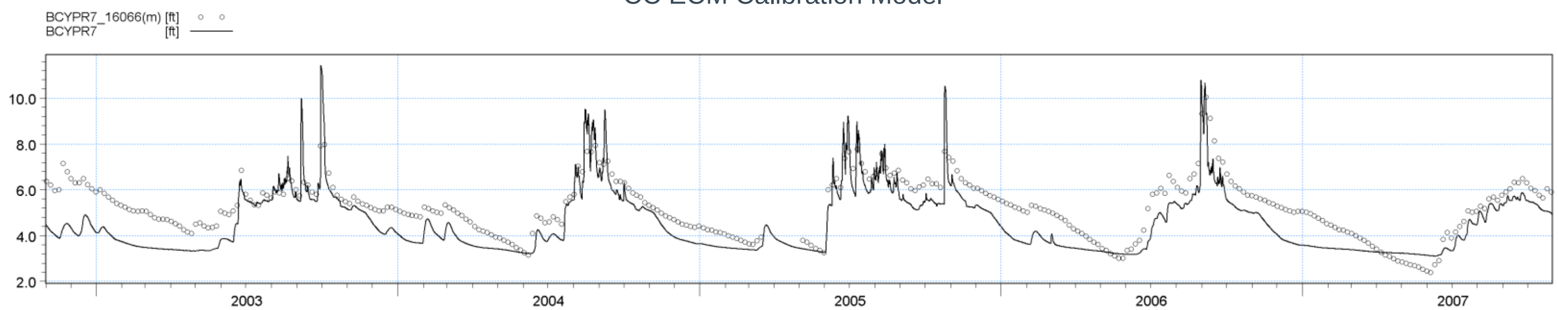


ME=0.118525
MAE=0.392399
RMSE=0.567544
STDres=0.555029
R(Correlation)=0.897313
R2(Nash_Sutcliffe)=0.619551

Figure 18. Comparison of Initial Local Scale Model Run to CC ECM Results Stage Monitoring Station BCYPR7

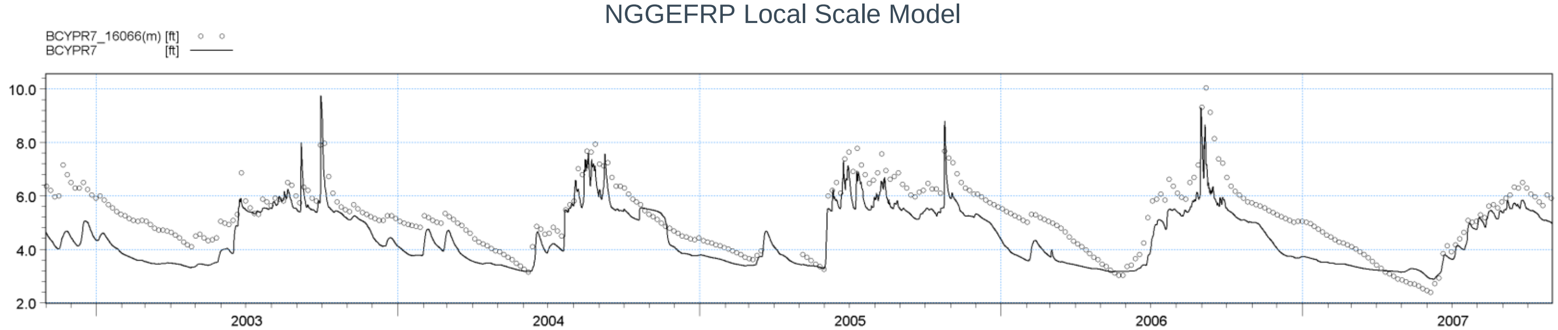
Station BCYPR7 (Miller Canal)

CC ECM Calibration Model



ME=0.701334
MAE=0.79541
RMSE=0.935429
STDres=0.618997
R(Correlation)=0.880642
R2(Nash_Sutcliffe)=0.43088

NGGEFRP Local Scale Model



ME=0.757345
MAE=0.797292
RMSE=0.933898
STDres=0.546438
R(Correlation)=0.89894
R2(Nash_Sutcliffe)=0.432742

In general, the comparison indicates that the initial run of the local scale NGGEFRP model produces very similar or better results than the 7-layer CC ECM model. The results for monitoring well C-1245 and stage station BCYPR7 are essentially equal for the two models. In the water table aquifer well C-953, the level of calibration has improved significantly as indicated by a mean error of -0.50 feet in the local scale model versus -1.44 feet in the 7-layer CC ECM. Statistical improvements are also noted for Well C-951 and stage station GOLD.W4_H.

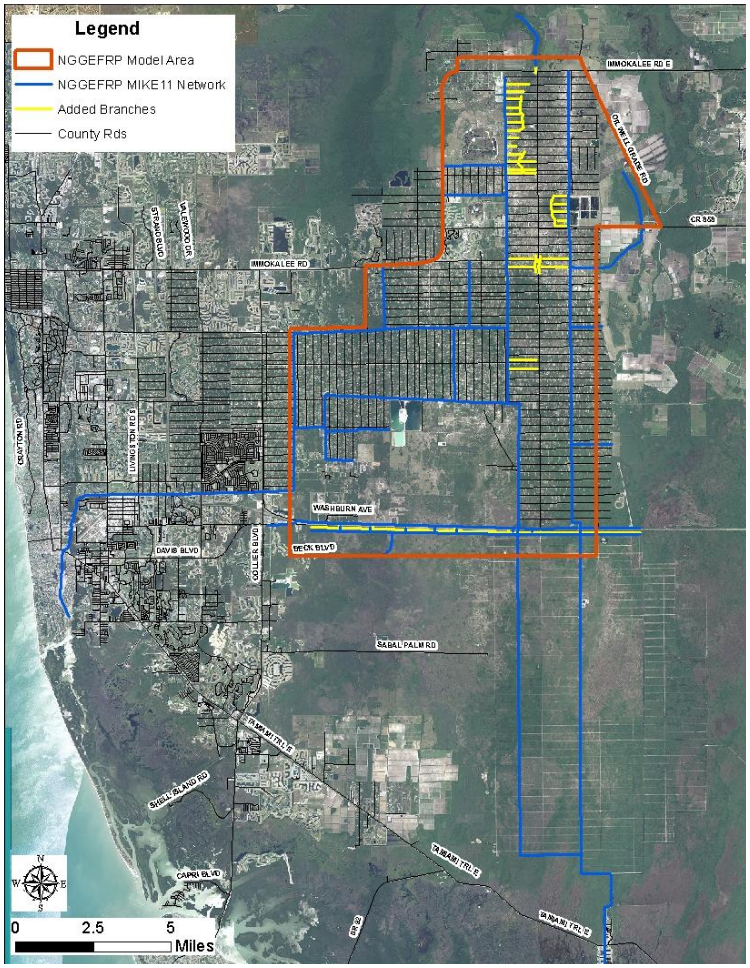
The comparison also indicates that the predicted peak stages in the canals are often one (1) or more feet lower in the local scale model than the larger scale CC ECM and more closely match the observed data. This is likely related to the more refined topography associated with 500-ft grid and suggests that more water is stored on the ground surface. The updated land use data may also reduce runoff since some lands that were simulated as low density urban in the CC ECM are now modeled as natural areas in the local scale model.

2.10. Add additional hydraulic features

Atkins staff inspected the roads in the Everglades Blvd corridor to identify cross-drains and locations where road inundation was likely to occur. Rapid survey methods were used to estimate the invert elevations of culverts crossing under roads in the study area. This information was used to update the MIKE 11 network to allow flow from north to south between the Separated Overland Flow areas in areas where overtopping was observed.

The revised MIKE 11 river network is shown in **Figure 19**.

Figure 19. North Golden Gate Estates Flowway Restoration Project Revised River Network



Based on the field work, the following culverts were added to the model:

- Two 36-inch culverts under Immokalee Rd near the intersection with Everglades Blvd.
- Four 18-inch culverts under 62nd Ave NE west of Everglades Blvd.
- Two 24-inch culverts under 52 Ave NE west of Everglades Blvd.
- Three 24-inch culverts under 48th Ave NE west of Everglades Blvd.
- Three 24-inch culverts under 39th Ave NE east of Everglades Blvd.
- Three 24-inch culverts under Randall Blvd at Everglades Blvd.
- Two 24-inch culverts under Everglades Blvd at Randall Blvd.

Broad crested weirs were added to the model to represent overtopping of roads at the following locations:

- On 66th Ave NE, west of Everglades Blvd in the Panther Walk area.
- On 64th Ave NE west of Everglades Blvd in the Panther Walk area.
- On 62nd Ave NE, west of Everglades Blvd in the Panther Walk area.
- On 60th Ave NE, west of Everglades Blvd in the Panther Walk area.
- On 41st Ave NE, east of Everglades Blvd in Winchester Head.
- On 39th Ave NE, east of Everglades Blvd in Winchester Head.
- On 37th Ave NE, east of Everglades Blvd in Winchester Head.

- On 35th Ave NE, east of Everglades Blvd in Winchester Head.
- On Randall Blvd, east of Everglades Blvd

In addition to the cross-drains that were added to the model, road side ditches were added in the areas of the model where the ditches directly drain wetland areas, or are associated with the major east-west roads. These areas include the Panther Walk and Winchester Head areas. Road side swales/ditches were included on the north side of the following roads:

- 70th Ave NE, west of Everglades Blvd.
- 68th Ave NE, west of Everglades Blvd.
- 66th Ave NE, west of Everglades Blvd.
- 64th Ave NE, west of Everglades Blvd.
- 62nd Ave NE, west of Everglades Blvd.
- 60th Ave NE, west of Everglades Blvd.
- 58th Ave NE, west of Everglades Blvd.
- 56th Ave NE, west of Everglades Blvd.
- 54th Ave NE, west of Everglades Blvd.
- 52nd Ave NE, west of Everglades Blvd.
- 50th Ave NE, west of Everglades Blvd.
- 48th Ave NE, west of Everglades Blvd.
- 47th Ave NE, west of Everglades Blvd.
- 41st Ave NE, east of Everglades Blvd.
- 39th Ave NE, east of Everglades Blvd.
- 37th Ave NE, east of Everglades Blvd.
- 35th Ave NE, east of Everglades Blvd.
- Oil Well Road, east of Everglades Blvd.
- Randall Blvd, east of Everglades Blvd.
- Randall Blvd, west of Everglades Blvd.
- 24th Ave NE, east of Everglades Blvd.
- 24th Ave NE, west of Everglades Blvd.
- Golden Gate Blvd, west of Everglades Blvd.
- 2nd Ave SE, west of Everglades Blvd.

Cross-sections for these road-side swale/ditches were cut from the DEM at intervals of approximately 500 feet. One cross-section in each swale was cut at the location estimated to have the highest bottom elevation. It was assumed that driveway culverts do not restrict discharges from the swale into the receiving canal. For stability purposes, a weir was specified in the model at the point of discharge from the each roadside swale to the primary receiving canal. The invert elevations of these weirs were set equal to the bottom elevation of the swale at the outfall.

2.11. Sub-Task 2.3 - Model Calibration

Several steps were completed during this sub-task.

Review of Observation Data: The observation data within the model domain was reviewed for completeness. During this evaluation, five wells were identified that were not included in the CC ECM. The wells are identified below and are shown of **Figure 13**. The observation data used in the calibration was defined as the model simulation period. The data was downloaded from either the DBHYDRO or USGS web pages and converted from the NGVD (1929) vertical datum to the NAVD (1988) vertical datum.

Well Name	Aquifer	Data Period
Well C-304	Lower Tamiami Aquifer	Jan 2002 – Mar 2004
Well C-956	Lower Tamiami Aquifer	Jan 2002 – Oct 2003
Well C-980	Water Table Aquifer	Jan 2002 – Sept 2004
Well C-976	Water Table Aquifer	Jan 2002 – Nov 2007
Well C-977	Lower Tamiami Aquifer	Jan 2002 – Nov 2007

Calibration Model Runs: Several additional model runs were completed during the process of adding the additional hydraulic data and to improve the model calibration for monitoring stations located near the model boundary. These runs are described in **Table 5** as follows:

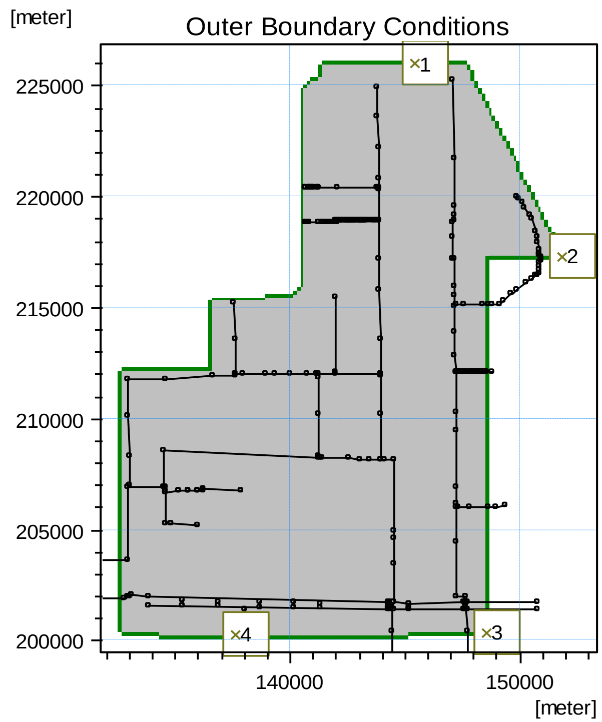
Table 5. Description of Model Runs

Run Name	Description
Initial Model Run	Model extracted from CC ECM. Includes 500 foot grid and modified land use inputs. All boundary conditions extracted from CC ECM results. Does not include any additional hydraulic features.
CC_EC_Rev3a_NGGE_Run2.SHE	Added monitoring wells C-304, C-956, C-976, C-977, and C-980; modified M11 leakage coefficient in coupling to 0.001 for segments of Miller and Faka Union Canals near I-75. Results showed no improvement in Groundwater or Surface water results in/near Miller and Faka Union Canals. Return coefficient to 0.0001 for Run 3
CC_EC_Rev3a_NGGE_Run3.SHE	Modify water table boundary conditions northeast, east and south of model (**Figure 20**). Measured data(1) was used to define the boundary condition. If the measured data did not cover full simulation period, then Julian year daily averages were calculated and applied to all years of simulation.
CC_EC_Rev3a_NGGE_Run4.SHE	Changed the end points of the water table boundary segments (**Figure 21**); added Lower Tamiami boundary conditions northeast, east and south of model Measured data(2) was used to define the boundary condition. If the measured data did not cover full simulation period, then Julian year daily averages were calculated and applied to all years of simulation. Set the boundary for Bonita Springs Marl to no flow as in CC ECM.

Run Name	Description
CC_EC_Rev3a_NGGE_Run5.SHE	Returned to boundary data extracted from CC ECM for WT and LT aquifers near the FU5 structure **(Figure 22)**. Utilized measured boundary data for northeast, southeast, and southern portions of model domain. Moved the location of the I-75 south canal north approximately 800 feet so that a single grid cell separates the north and south canals. Modify flood code file to compensate for the change.
CC_EC_Rev3a_NGGE_Run6.SHE	Added branches and structures for Panther Walk and Winchester Head. Updated Separated Flow Areas to provide breaks at known structures in the flow way. Extended Corkscrew Trib2 branch to the south and added two 36-inch culverts underneath Immokalee Rd.
CC_EC_Rev3a_NGGE_Run7.SHE	Hydroperiod results for Run 6 appear to be inaccurate for southern part of the Everglades Blvd corridor. Added additional Separated Overland Flow zones in the southern portion of the study area between Miller and Faka Union Canals.
CC_EC_Rev3a_NGGE_Run8.SHE	Hydroperiod results are influenced by the eastern boundary conditions. Modified end points of eastern boundary (**Figure 23**) to use results of CC ECM over a larger area.
CC_EC_Rev3a_NGGE_Run9.SHE	Added roadside swales and ditches in Panther Creek and Winchester Head areas of NGGE and along Randall and Golden Gate Blvds.

Boundary Conditions: As noted in **Table 5**, ground boundary condition input files were developed from observation data taken from monitoring wells located in the northeast, southeast and southern portions of the model. Monitoring data was converted from the National Geodetic Vertical Datum (1929) to the North American Vertical Datum (1988) to be consistent with the model.

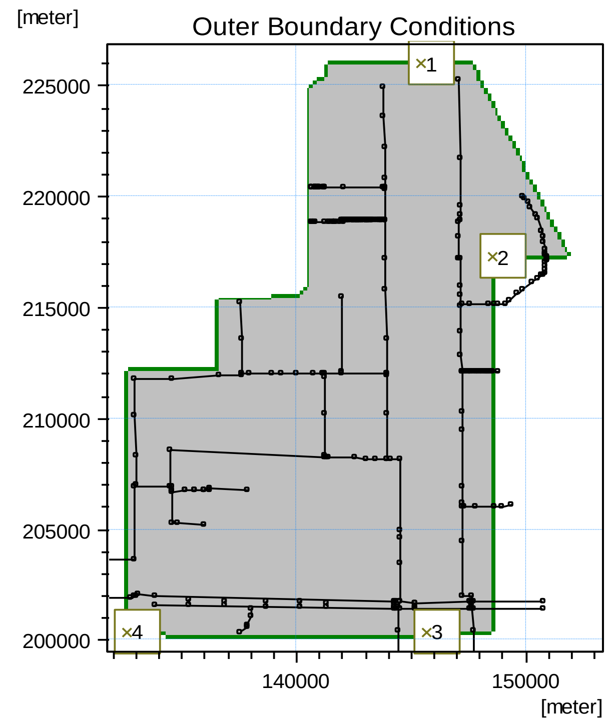
Figure 20. Run 3 Boundary Conditions



Run 3 Boundaries

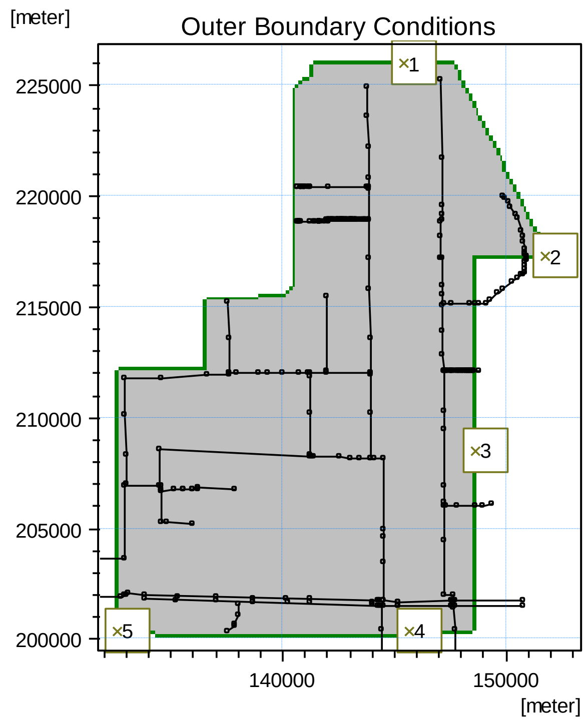
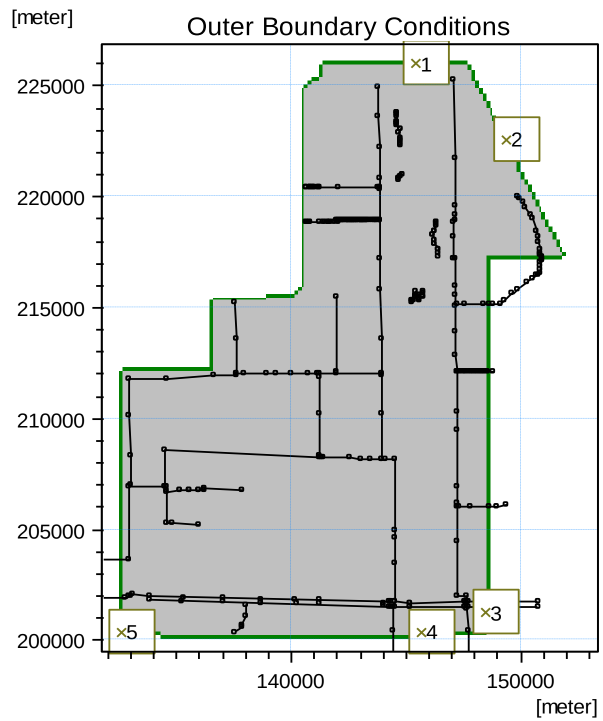
Segment from 1 - 2: Julian Day from Well C-1245
Segment from 2 – 3: Julian Day from SGT1W4 and LuckW_GW
Segment from 3 – 4: Julian Day from SGT1W1 and SGT1W2
Segment from 4 – 1: Extracted from CC ECM

Figure 21. Run 4 Boundary Conditions



Run 4 Boundaries

Segment from 1 - 2: Julian Day from Well C-1245
Segment from 2 – 3: Julian Day from SGT1W4 and LuckW_GW
Segment from 3 – 4: Julian Day from SGT1W1 and SGT1W2
Segment from 4 – 1: Extracted from CC ECM

Figure 22. Run 5 Boundary Conditions	Figure 23. Run 8 Boundary Conditions
	
Run 5 Boundaries Segment from 1 - 2: Julian Day from Well C-1245 Segment from 2 – 3: Extracted from CC ECM Segment from 3 – 4: Julian Day from SGT1W4 and LuckW_GW Segment from 4 – 5: Julian Day from SGT1W1 and SGT1W2 Segment from 5 – 1: Extracted from CC ECM	Run 8 Boundaries Segment from 1 - 2: Julian Day from Well C-1245 Segment from 2 – 3: Extracted from CC ECM Segment from 3 – 4: Julian Day from SGT1W4 and LuckW_GW Segment from 4 – 5: Julian Day from SGT1W1 and SGT1W2 Segment from 5 – 1: Extracted from CC ECM

Northeast: A Julian day time series was developed from data collected from monitoring well C-1245. Data from other nearby wells (HF3_G and HF4_G) was not included because those wells do not show the effect of pumping from irrigation wells near the model boundary.

Southeast: A Julian day time series was developed from measured data collected in wells SGT1W4 and LuckW_GW.

South: A Julian day time series was developed from measured data collected in wells SGT1W1 and SGT1W2.

Surface water boundary conditions for existing branches were developed by extracted predicted time series of stage from the CC ECM MIKE 11 model results for the following branches:

- CorkscrewTrib2 at location 10302 feet
- I75N-3 at location 10171 feet
- I75S-3 at location 10171 feet
- HendersonCr at location 9390 feet
- BelleMeade-1 at location 4212 feet

Calibration Results: Water budgets were calculated for each water year of the model simulation and then averaged to obtain an average water budget for five (5) water years included in the simulation. Each water year is defined as being from November 1 through October 31, so the 2003 water year is from November 1, 2002 – October 31, 2003. **Table 6** shows the water budgets for the NGGE_Run9 model. The results are similar to those generated by the CC ECM model in the Golden Gate Watershed (Atkins, 2011), with Evapotranspiration (ET) equaling slightly more than 60 percent of rainfall.

Table 6. Annual Water Year Water Budgets for NGGE_Run9 Model

Water Year	Inputs				Output						Storage Change
	Rainfall	Irrigation	Overland Boundary Inflow	Aquifer Boundary Inflow	ET	Runoff	Baseflow to River	Pumping	Overland Boundary Outflow	Aquifer Boundary Outflow	
2003	63.39	2.05	0.00	10.04	36.02	2.56	18.27	7.83	1.30	5.55	1.22
2004	63.46	2.64	0.00	9.72	36.57	3.03	17.01	8.58	1.38	5.87	0.39
2005	72.80	2.68	0.00	9.92	37.56	3.66	20.31	8.90	1.69	6.02	2.68
2006	55.35	4.29	0.00	10.04	36.97	2.60	15.83	10.35	0.87	5.39	-3.27
2007	45.75	4.17	0.00	8.43	37.20	0.31	7.40	10.16	0.20	3.98	-0.71
Average	60.15	3.17	0.00	9.63	36.87	2.43	15.76	9.17	1.09	5.36	0.06

Tables 7 and 8 summarize the calibration statistics for the groundwater and surface water monitoring stations in both the 7-layer CC ECM and the NGGE_Run9 model. Individual calibration plots for each of stations listed in **Tables 7 and 8** are found in Appendix B.

The calibration statistics indicate that the NGGEFRP local scale model is generally better calibrated than the CC ECM calibration model in the canals, the Water Table aquifer, and the Lower Tamiami aquifer. In the canals, the average Mean Absolute Error (MAE) has decreased from 0.826 feet to 0.795 feet and the average Correlation (R) value has increased from 0.771 to 0.797. Both parameters indicate an improved level of statistical calibration.

For the Water Table aquifer, the CC ECM considered four (4) wells while the NGGEFRP model identified six (6) wells. For the four (4) wells defined in the CC ECM, the average MAE decreased from 1.655 feet to 0.903 feet and the average R value increased from 0.914 to 0.940. If the six wells in the NGGEFRP are considered, then the average MAE decreased from 1.655 feet to 1.01 feet and the R value increased from 0.914 to 0.929. Both parameters indicate an overall improvement in calibration of the aquifer.

It is noted that only one of the Water Table monitoring wells is located in the center of the model area. The ME in well C-953 improves by approximately one (1) foot to a value of -0.47 and the MAE improves by approximately 0.6 feet to a value of 0.84. In this well, the statistical results (ME is negative) indicate that the model generally over-predicts the groundwater head elevation. However, the calibration plot (**Figure 24)** suggests that the NGGEFRP model tends to slightly over-predict head elevation during the rainy season, while under-predicting the groundwater level during the dry season. This pattern of predicted results must be considered when evaluating alternative management strategies. All other Water Table monitoring wells are located on, or near the model boundary and may not be indicative of ground water elevations in the NGGEFRS area.

In the Lower Tamiami aquifer, the CC ECM included two (2) wells, while the NGGEFRP model considered five (5) wells with the model domain. The results indicate that the NGGEFRP has an average ME that is more than two (2) feet less than for the CC ECM. The MAE improved by more than 1.6 feet and the R value increased from 0.6 to 0.9. In many of the wells, the average mean error is negative indicating that the model over-predicts the head elevation in the aquifer throughout the year. This can be seen in monitoring well C-951 (**Figure 25**).

Because there are zero (0) monitoring well directly located within the area of the proposed NGGE Flowway Restoration Project, it is assumed that the head elevation in the Water Table aquifer is controlled by the stage in the canal network and that calibration of surface water stations is very important. **Figure 26 – 31** show calibration plots at several surface water stations in the model domain.

Statistically, the surface water stations are well calibrated with an average ME across all stations less than 0.15 feet and a MAE less than 0.80 feet. In the Golden Gate Main Canal, the average ME of monitoring stations is 0.02 feet and the MAE is 0.43 feet with a correlation greater than 0.80. This indicates that the groundwater boundary conditions on the eastern side of the model that may affect results in the canals are appropriate for the model.

The surface water stations with the poorest level of calibration are found in the Faka Union Canal between stations FU-4 and FU-5. Here the statistical results indicate that the ME is greater than 1.0 feet and the MAE is greater than 1.7 feet. **Figure 30** shows the results at station FU4_H. This plot indicates that the large error occurs primarily during the dry season, in the months of December through May. Predicted wet season stages are much closer to observed stages suggesting that the model will be useful for assessing wet season conditions in the study area.

Table 7. Calibration Statistics for Groundwater Monitoring Stations

Station ID	Location	Statistic	7-layer CC ECM	NGGE_Run9
Well C-953	Water Table Aquifer	Mean Error (ft)	-1.44	-0.471
		Mean Absolute Error (ft)	1.46	0.835
		Correlation (R)	0.927	0.932
Well SGT1W1	Water Table Aquifer	Mean Error (ft)	-1.988	-1.062
		Mean Absolute Error (ft)	1.990	1.066
		Correlation (R)	0.885	0.918
Well SGT1W2	Water Table Aquifer	Mean Error (ft)	-1.956	-1.298
		Mean Absolute Error (ft)	1.958	1.303
		Correlation (R)	0.919	0.926
Well SGT1W4	Water Table Aquifer	Mean Error (ft)	-0.907	0.338
		Mean Absolute Error (ft)	1.210	0.409
		Correlation (R)	0.923	0.985
Well C-980	Water Table Aquifer	Mean Error (ft)	-	-1.022
		Mean Absolute Error (ft)	-	1.384
		Correlation (R)	-	0.922
Well C-976	Water Table Aquifer	Mean Error (ft)	-	-0.957
		Mean Absolute Error (ft)	-	1.065
		Correlation (R)	-	0.892
Well C-951	Lower Tamiami Aquifer	Mean Error (ft)	-3.513	-2.870
		Mean Absolute Error (ft)	3.513	2.870
		Correlation (R)	0.932	0.928
Well C-1245	Lower Tamiami Aquifer	Mean Error (ft)	-5.131	0.270
		Mean Absolute Error (ft)	5.479	2.846
		Correlation (R)	0.302	0.763
Well C-304	Lower Tamiami Aquifer	Mean Error (ft)	-	-4.396
		Mean Absolute Error (ft)	-	4.396
		Correlation (R)	-	0.920
Well C-956	Lower Tamiami Aquifer	Mean Error (ft)	-	-3.502
		Mean Absolute Error (ft)	-	3.502
		Correlation (R)	-	0.979
Well C-977	Lower Tamiami Aquifer	Mean Error (ft)	-	-0.585
		Mean Absolute Error (ft)	-	0.695
		Correlation (R)	-	0.952
Well C-948	Mid-Hawthorn Aquifer	Mean Error (ft)	24.213	24.680
		Mean Absolute Error (ft)	24.213	24.68
		Correlation (R)	0.193	0.181
Well C-974	Mid-Hawthorn Aquifer	Mean Error (ft)	25.500	25.614
		Mean Absolute Error (ft)	25.500	25.614
		Correlation (R)	0.506	0.503

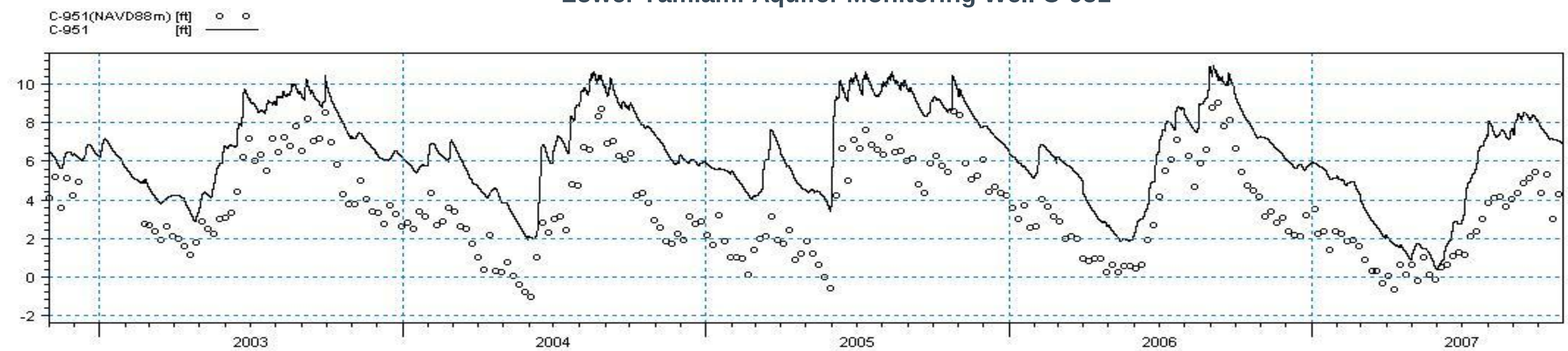
Table 8. Calibration Statistics for Surface Water Monitoring Stations

Station ID	Location	Statistic	7-layer CC ECM	NGGE_Run9
MLRI75_H	Miller Canal at I-75; headwater	Mean Error (ft)	0.795	0.869
		Mean Absolute Error (ft)	0.833	0.872
		Correlation (R)	0.865	0.870
BCYPR7	Miller Canal near 28th Ave SE	Mean Error (ft)	0.701	0.823
		Mean Absolute Error (ft)	0.795	0.844
		Correlation (R)	0.881	0.899
FAKI75	Faka Union Canal at I-75; headwater	Mean Error (ft)	0.586	-0.517
		Mean Absolute Error (ft)	0.611	0.565
		Correlation (R)	0.751	0.824
FU4_H	Headwater of Structure FU-4	Mean Error (ft)	-1.684	-1.267
		Mean Absolute Error (ft)	1.838	1.456
		Correlation (R)	0.737	0.796
FU4S_H	Headwater of Structure FU-4	Mean Error (ft)	-1.574	-1.133
		Mean Absolute Error (ft)	2.230	2.192
		Correlation (R)	0.646	0.670
FU4S_T	Tailwater of Structure FU-4	Mean Error (ft)	-0.464	-0.238
		Mean Absolute Error (ft)	0.675	0.473
		Correlation (R)	0.929	0.953
FU5_H	Headwater of Structure FU-5	Mean Error (ft)	0.214	0.368
		Mean Absolute Error (ft)	0.575	0.642
		Correlation (R)	0.647	0.658
FU5_T	Tailwater of Structure FU-5	Mean Error (ft)	-1.480	-1.062
		Mean Absolute Error (ft)	1.823	1.602
		Correlation (R)	0.581	0.662
GOLD.W3_T	Tailwater of Structure GG-3	Mean Error (ft)	-0.131	-0.117
		Mean Absolute Error (ft)	0.179	0.624
		Correlation (R)	0.848	0.799
GOLD.W3_H	Headwater of Structure GG-3	Mean Error (ft)	-0.440	-0.017
		Mean Absolute Error (ft)	0.584	0.512
		Correlation (R)	0.855	0.880
GOLD.W4_H	Headwater of Structure GG-4	Mean Error (ft)	-0.307	0.130
		Mean Absolute Error (ft)	0.394	0.399
		Correlation (R)	0.871	0.896
GOLD.W4A	Headwater of CYP-1 Structure on Cypress Canal near 25th St NW	Mean Error (ft)	-0.271	0.137
		Mean Absolute Error (ft)	0.380	0.411
		Correlation (R)	0.837	0.881
GOLD.W5_T	Tailwater of Structure GG-5	Mean Error (ft)	-0.374	-0.004
		Mean Absolute Error (ft)	0.450	0.425
		Correlation (R)	0.864	0.886
GOLD.W5_H	Headwater of Structure GG-5	Mean Error (ft)	0.054	-0.001
		Mean Absolute Error (ft)	0.190	0.194
		Correlation (R)	0.480	0.549

Figure 24. NGGEFRP_Run9 Results
Water Table Aquifer Monitoring Well C-953

ME=-0.471025
MAE=0.835104
RMSE=0.994861
STDres=0.87629
R(Correlation)=0.932897
R2(Nash_Sutcliffe)=0.567868

Figure 25. NGGEFRP_Run9 Results
Lower Tamiami Aquifer Monitoring Well C-951

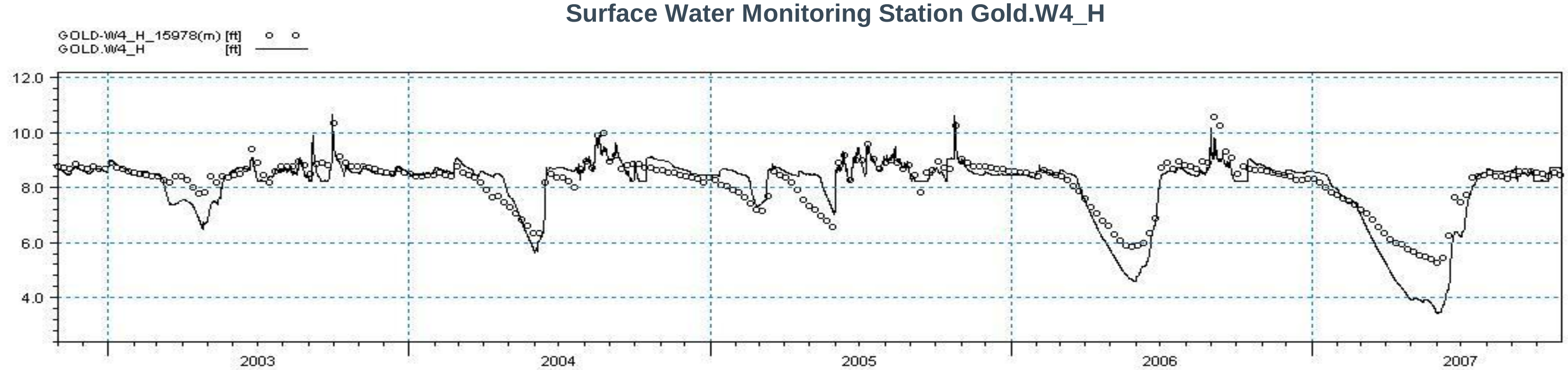


ME=-2.86914
MAE=2.8703
RMSE=3.00612
STDres=0.897092
R(Correlation)=0.928423
R2(Nash_Sutcliffe)=-0.68464

Figure 26. NGGEFRP_Run9 Results
Surface Water Monitoring Station Gold.W5_H

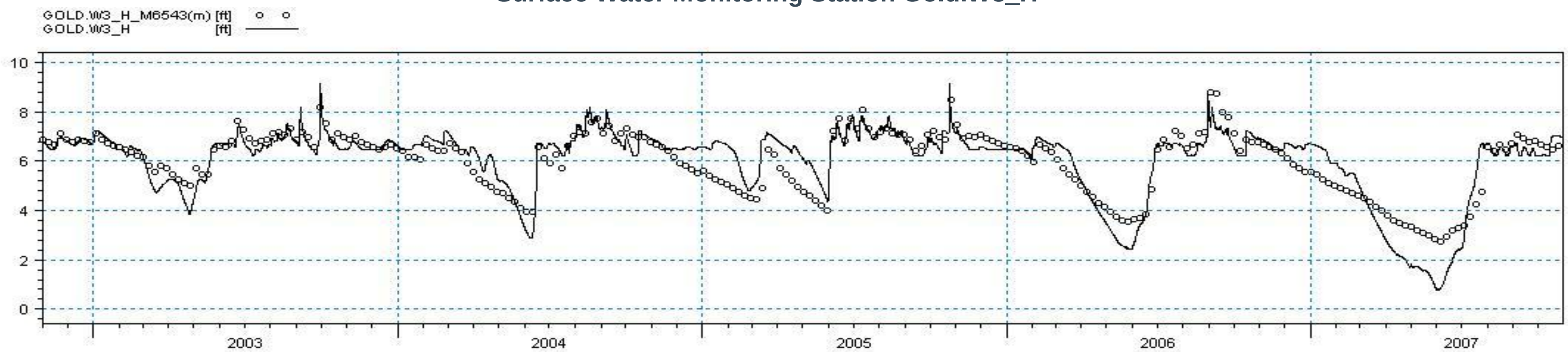
ME=-0.00112448
MAE=0.193685
RMSE=0.30194
STDres=0.301938
R(Correlation)=0.548949
R2(Nash_Sutcliffe)=-0.0165576

Figure 27. NGGEFRP_ Run9 Results
Surface Water Monitoring Station Gold.W4_H



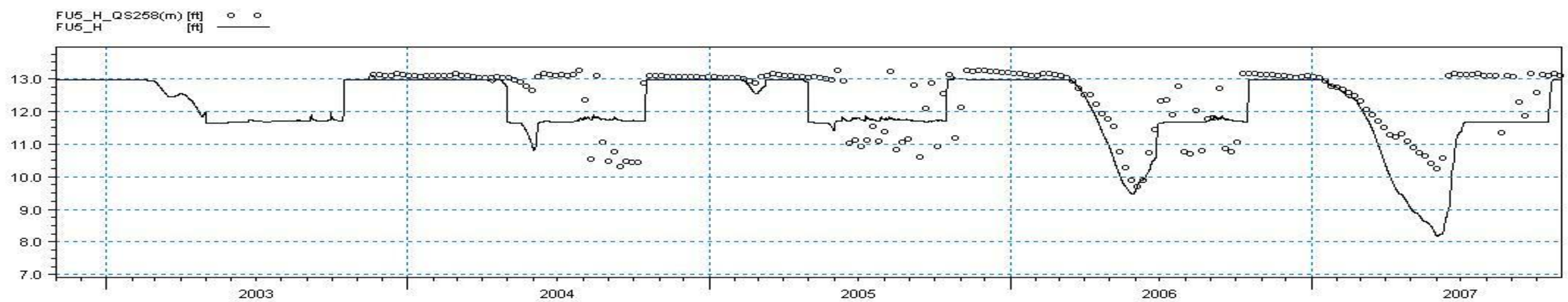
ME=0.13014
MAE=0.399132
RMSE=0.578673
STDres=0.56385
R(Correlation)=0.896389
R2(Nash_Sutcliffe)=0.604483

Figure 28. NGGEFRP_Run9 Results
Surface Water Monitoring Station Gold.W3_H



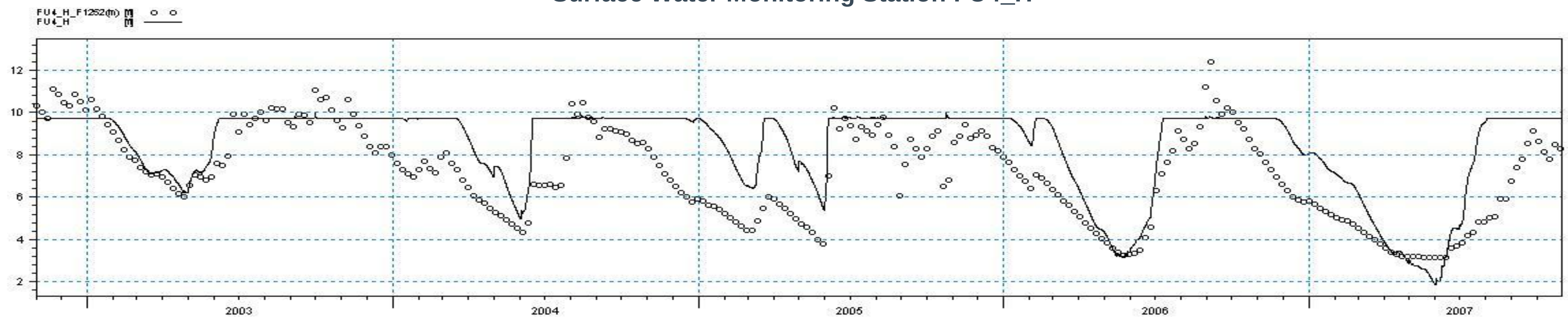
ME=-0.0170695
MAE=0.511934
RMSE=0.675912
STDres=0.675696
R(Correlation)=0.879784
R2(Nash_Sutcliffe)=0.701311

Figure 29. NGGEFRP_Run9 Results
Surface Water Monitoring Station FU5_H



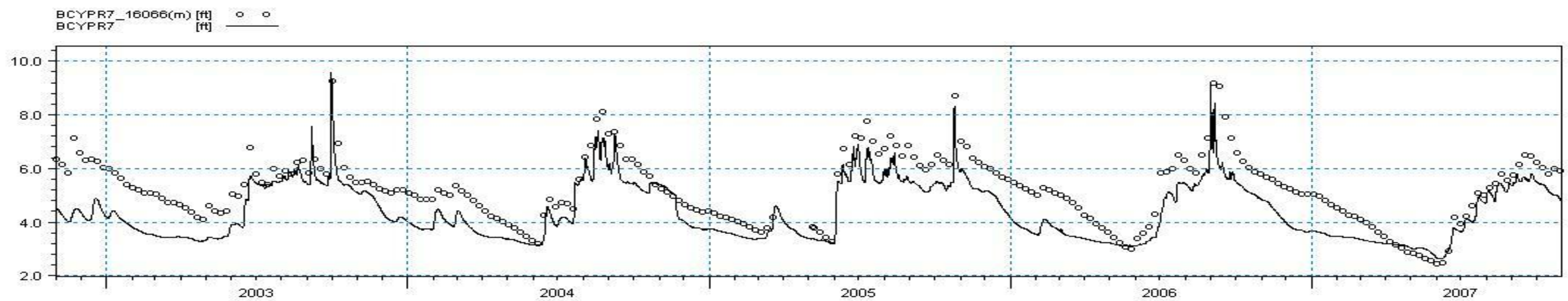
ME=0.368381
MAE=0.642265
RMSE=0.930761
STDres=0.854758
R(Correlation)=0.657871
R2(Nash_Sutcliffe)=0.0958454

Figure 30. NGGEFRP_Run9 Results
Surface Water Monitoring Station FU4_H



ME=-1.26729
MAE=1.45604
RMSE=1.84126
STDres=1.33574
R(Correlation)=0.795993
R2(Nash Sutcliffe)=0.272281

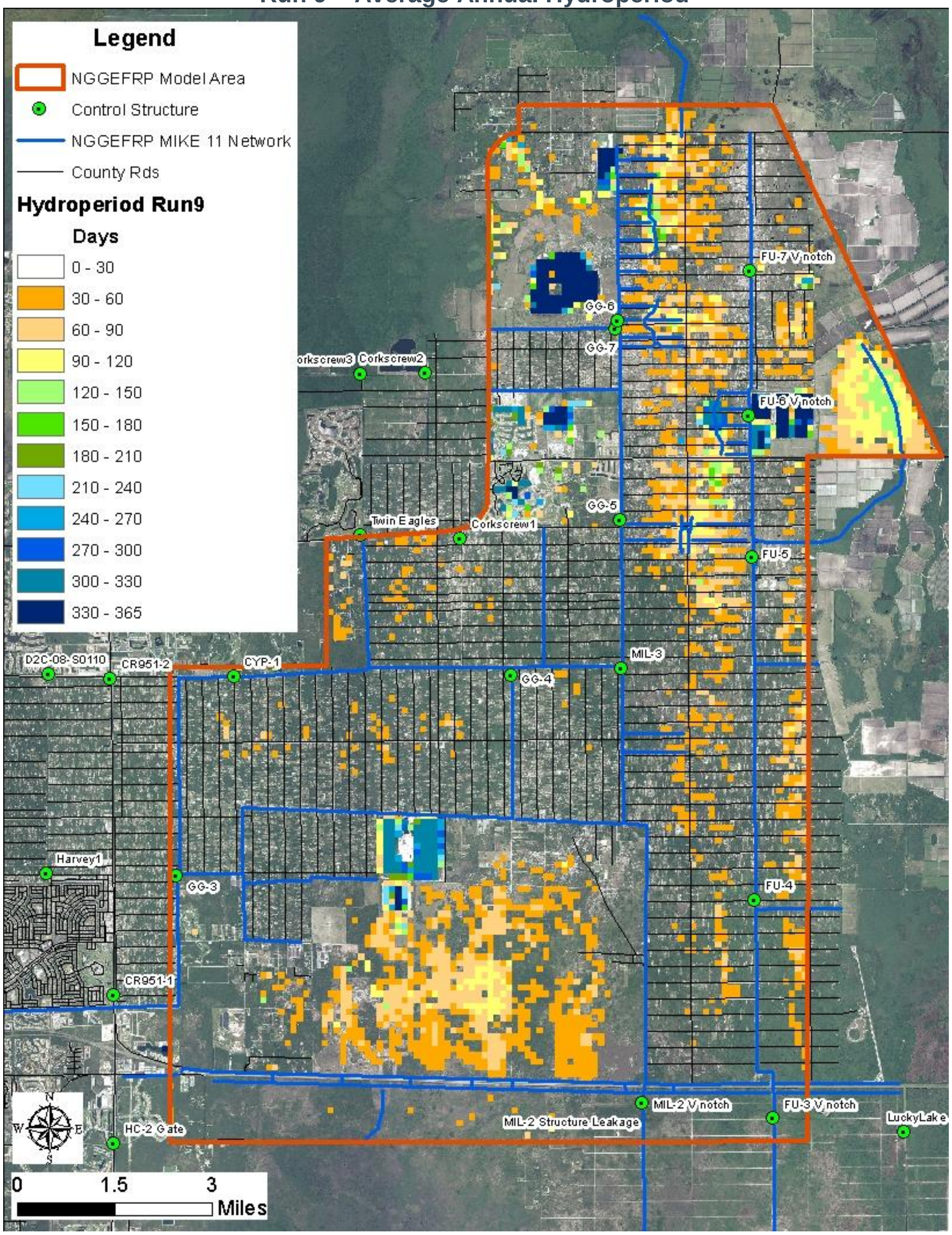
Figure 31. NGGEFRP_Run9 Results
Surface Water Monitoring Station BCYPR7



ME=0.823439
MAE=0.844523
RMSE=0.9894
STDres=0.548508
R(Correlation)=0.898698
R2(Nash_Sutcliffe)=0.363313

Figure 32 shows the hydroperiod results from Run 9. The hydroperiod was determined using the "Percent Exceedance" tool in the MIKE Zero Toolbox. The tool was used to calculate the percentage of time that the overland depth of water was greater than one (1) inch in each cell for each water year. This value was selected in order to eliminate counting results generated during dry season rainfall events. The results for each year were multiplied by 365 days to estimate the annual hydroperiod. The results for each year were then averaged to produce the Average Annual Hydroperiod map shown in **Figure 32**.

Figure 32. North Golden Gate Estates Flowway Restoration Project
Run 9 – Average Annual Hydroperiod



The predicted hydroperiod results shown in **Figure 32** indicate a hydroperiod of 6 – 8 months in the Panther Walk area which is consistent with the cypress land cover in that area. The results shown that Winchester Head has a predicted hydroperiod of 10 – 11 months, which is longer than would be expected for the cypress and swamp forest land cover. It is likely that the hydroperiod results in this area are influenced by the canal elevation controlled by the FU-6 structure. It is also possible that the model is over-predicting groundwater head elevation in the Water Table aquifer during the wet season. This must be considered when evaluating the benefits of proposed projects.

The results in **Figure 32** also suggest that the water surface elevation in the canals affects the hydroperiod of the adjacent lands. The lands north of the GG-5 and FU-5 structures generally have a longer hydroperiod than the lands to the south where the predicted hydroperiod is typically less than 30 days. This is likely determined by the operational control elevation of the structures. Mike Duever, a respected wetland scientist in southwest Florida was asked to review the average annual hydroperiod map. He stated (2012) that, “The hydroperiod map looks excellent to me. It shows the effects of canals very nicely.”

2.12. Summary and Conclusions

A local scale MIKE SHE – MIKE 11 model was extracted from the CC ECM to evaluate the potential benefits of the North Golden Gate Estates Flowway Restoration Project. The local scale model is based on a 500-ft grid cell and includes a more detailed representation of important hydraulic features within the flowway. The following conclusions can be drawn about the usefulness of the model to evaluate proposed projects.

- The calibration statistics for the local scale model are very similar to; or better than the statistics for the CC ECM. In addition, the predicted hydroperiod map of the study area is considered to be very good. These results indicate that the model is a good tool to evaluate potential projects in the flowway.
- In the center of the model area the results indicate a general over-prediction of head elevation in the Water Table aquifer of approximately 5.5 inches. This may influence the hydroperiod in wetland areas and should be considered when evaluating project benefits.
- The water surface elevations in the canals are generally well calibrated and control the groundwater head elevation. The wet season stage is managed by control structures and influences the hydroperiod of adjacent lands. This also should be considered when evaluating project benefits.

3. Task 3 – Alternatives Analysis

3.1. Introduction and Objective

This technical memorandum addresses Sub-Task 2.4 through 2.7 portions of Task 2 of the Scope of Work (SOW). Sub-task 2.6 is to develop and evaluate a total of four scenarios that focus on connectivity and restoration of wetlands in the Northern Golden Gate Estates Flowway Restoration Project Area (NGGEFRPA). This memorandum describes each of the four scenarios evaluated and presents the hydrologic and hydraulic analysis for each scenario. Comparative results of design storm simulations are presented for Scenario 4 which includes all recommended improvements.

3.2. Develop Modified Existing Conditions Scenario

Prior to evaluating the water management strategies, a modified Existing Conditions model was developed which includes new water control features implemented in the Golden Gate canal network. This model was developed from the calibration model described in Sub-Tasks 2.1 – 2.3 and includes the following modifications to the MIKE 11 component of the calibrated model:

- Moved the GG-3 structure and modified the structure operations to be consistent with the new constructed replacement structure.
- Removed the Miller-2 structure and replaced it with a pump station that has a total maximum capacity of 1,250 cubic feet per second (cfs) as defined in the Picayune Strand Restoration Project.
- Removed the FakaUnion-3 structure and replaced it with a pump station that has a total maximum capacity of 2,650 cubic feet per second (cfs) as defined in the Picayune Strand Restoration Project.
- Removed the culvert on the Miller Canal at 38th Avenue SE.
- Modified the culverts under 28th Avenue SE to allow greater conveyance capacity.
- Modified the dimensions and structure operations of the Miller-3 structure to match the interim procedures defined by the South Florida Water Management District (SFWMD).
- Modified the dimensions and structure operations of the Golden Gate Canal-6 structure to correspond with the replacement structure.
- Modified the dimensions and structure operations of the Golden Gate Canal-7 structure to correspond with the replacement structure.

In addition to the changes in the MIKE 11 network, the separated overland flow input file was modified to represent the effect of building the extension of Wilson Blvd. south toward I-75. This change will prevent east-west overland flow along the road corridor.

The modified existing conditions model predicts that the hydroperiod in the Golden Gate Estates area between the Golden Gate Main/Miller Canal and the Faka Union Canal will generally be shorter than that predicted in the calibration model. This is most evident north of Oil Well Rd in the Panther Walk area and is shown in **Figures 33 and 34**. The predicted hydroperiod in the North Bell Meade area, after the modifications to the MIKE 11 network, is very similar to that predicted in the calibration model.

3.3. Identify Water Management Scenarios

This section describes the four proposed strategies (scenarios) to be considered during this project. Due to the presence of the primary roads and canals in the study area, the study area is effectively divided into three distinct sub-areas (shown in **Figure 35**). A brief description of each scenario is provided below. Detailed descriptions are presented in later sections of this document.

Scenario 1. This alternative focuses on Area 1 (North Belle Meade) and considers the diversion of water from the Golden Gate Main Canal into a spreader system that will direct water into the North Belle Meade area north of I-75. Two options were considered for this scenario. Option 1 assumed an 800 cfs pump station. Option 2 considered a 400 cfs pump station.

Scenario 2. This alternative in Area 2 is the portion of study area north of Oil Well Rd between the Golden Gate and Faka Union Canals. This scenario focuses on adding cross-drains under existing roads to improve the interconnection of the wetlands with the goal of extending the hydroperiod and increasing groundwater recharge.

Scenario 3. This alternative evaluates the effect of adding cross-drains under existing roads to increase the connectivity of wetland systems from Oil Well Rd to I-75 between the Golden Gate/Miller Canal and the Faka Union Canal and includes a small area west of the Miller Canal on either side of Golden Gate Blvd.

Scenario 4. This scenario combines the features of Scenarios 1, 2, and 3 in order to predict the cumulative effect of all proposed improvements.

Figure 33. NGGE Flowway Restoration Project Calibrated Model Predicted Hydroperiod

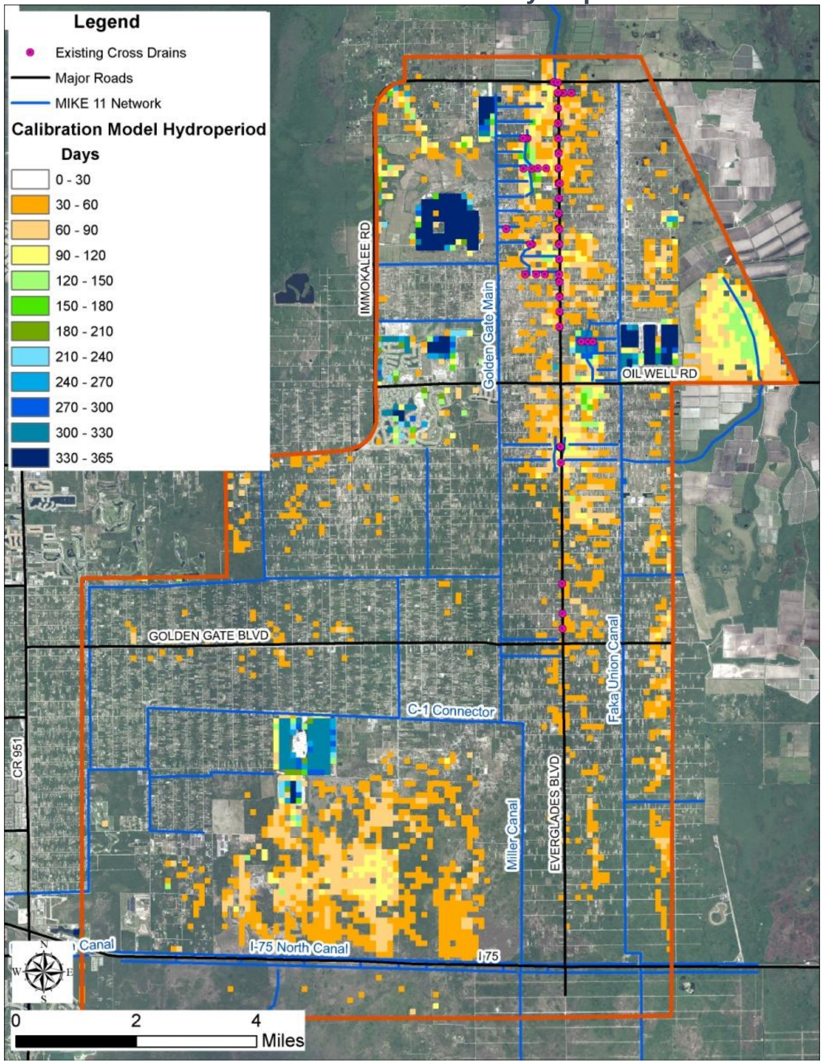


Figure 34. NGGE Flowway Restoration Project Modified Existing Conditions Predicted Hydroperiod

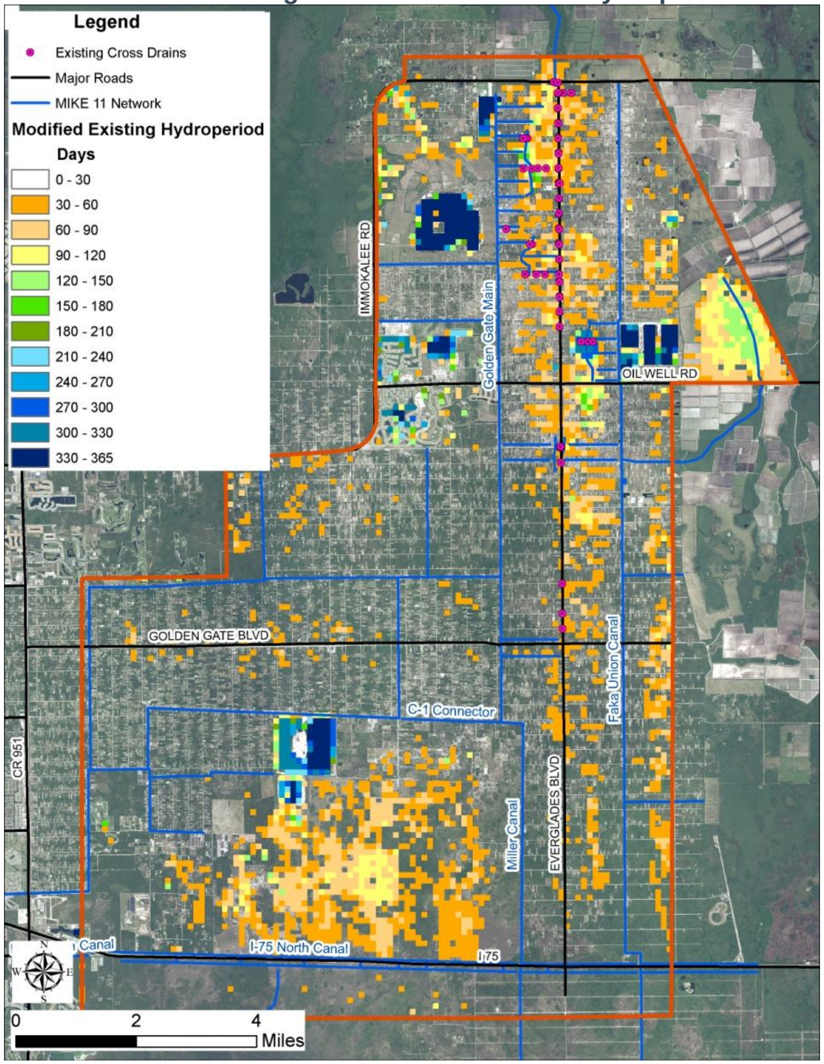
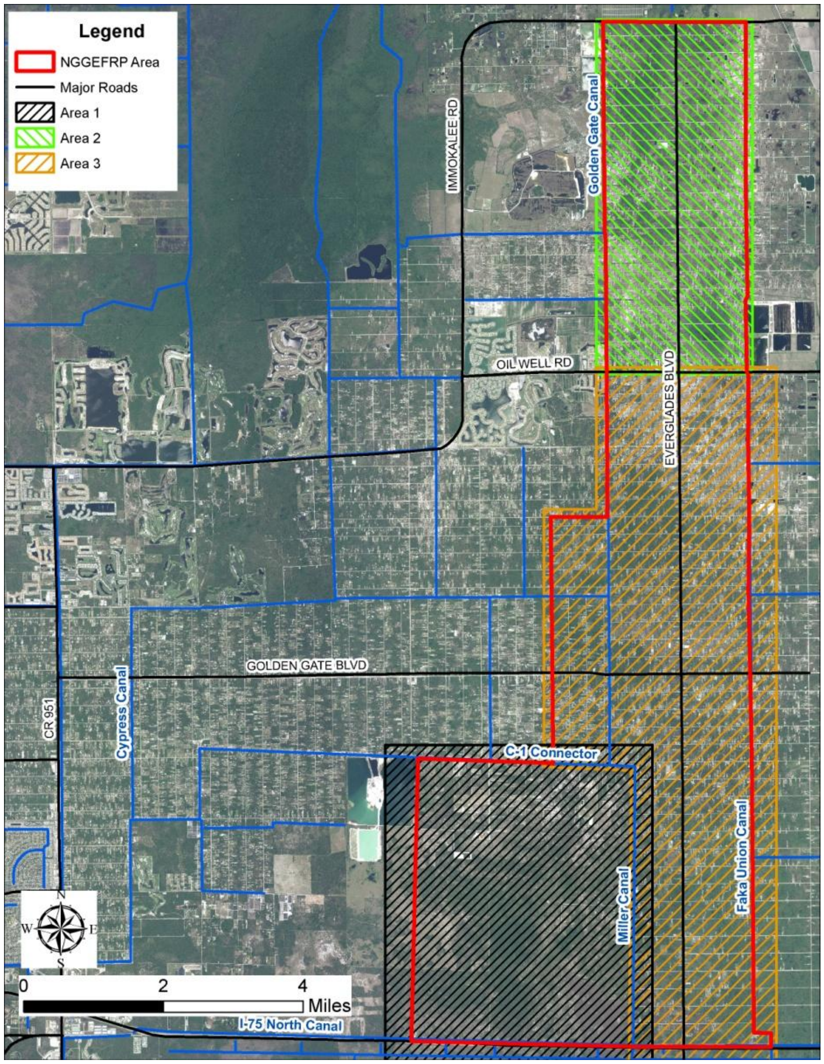


Figure 35. North Golden Gate Estates Flowway Restoration Project Scenario Areas



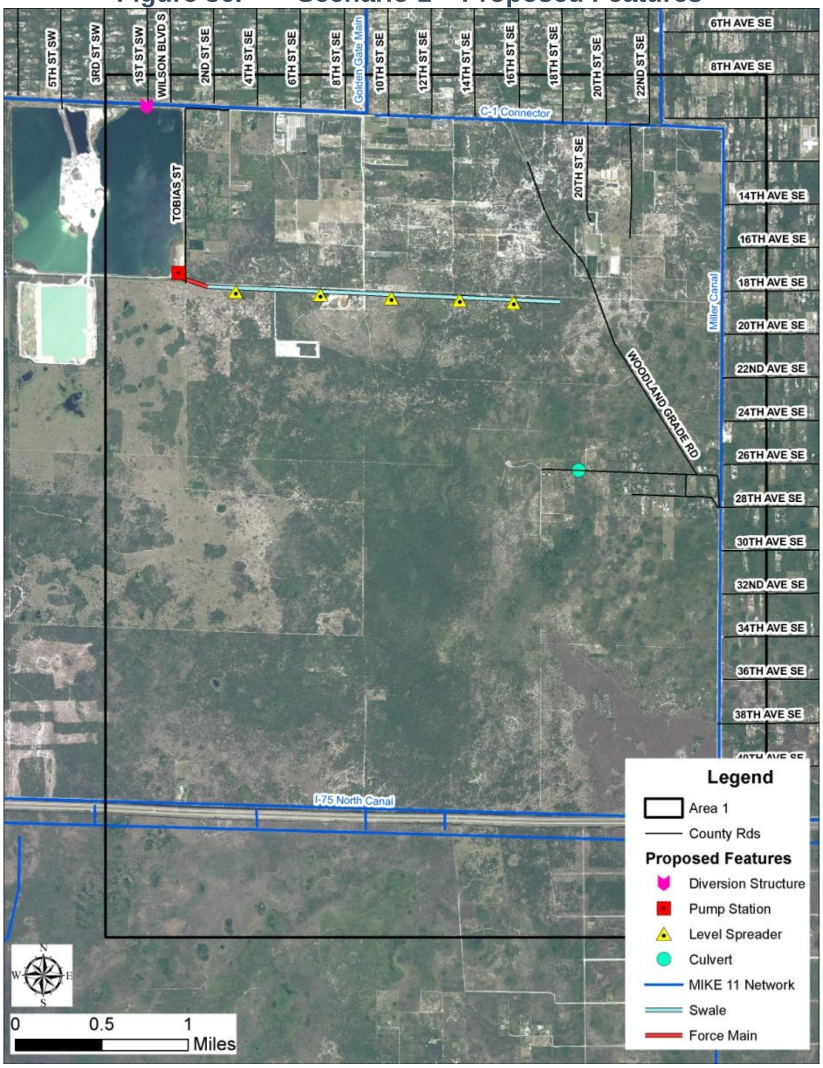
3.4. Scenario 1 – North Belle Meade Spreader Swale

This section describes the scenario evaluated for the North Belle Meade area. The objective is to divert water from the Golden Gate Main Canal during periods of high flow.

3.4.1. Description of Scenario

The Area 1 project scenario considers implementation of the North Belle Meade Spreader Swale which was identified as Element No. 6 in the Belle Meade Stormwater Management Master Plan (Parsons, 2006) and recommended for implementation in the Collier County Watershed Management Plan (Atkins, 2011). The Parsons report specified a system that diverts water from the Golden Gate Main Canal, at a maximum rate of 400 cfs, into the existing mine pits and then into the proposed spreader swale. The Collier County Watershed Management Plan recommended a smaller pump station (200 cfs) that moved water directly from the Golden Gate Main Canal to the proposed spreader swale. In both cases, it is expected that this facility would be used primarily during the rainy season to divert water from the Golden Gate – Naples Bay watershed into the Rookery Bay watershed. The diverted water would be used to rehydrate wetlands in the North Belle Meade area and to provide additional flows to the Rookery Bay and Ten Thousand Islands estuaries. Two options, based on the Parson's project configuration, were considered for this scenario. The general configuration of the proposed project for this evaluation includes the elements shown in **Figure 36.**

Figure 36. Scenario 1 – Proposed Features



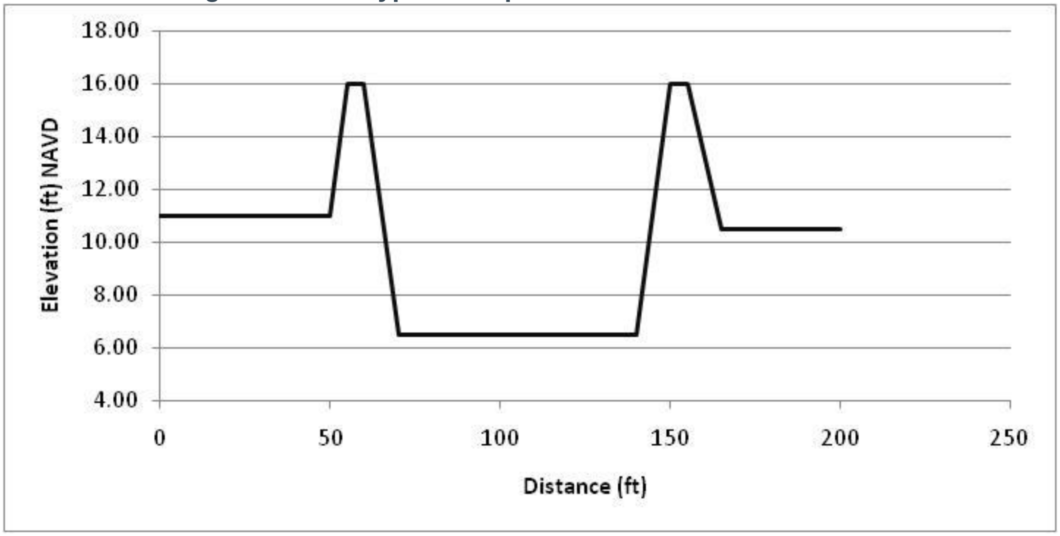
- **Pump Station.** A pump station will be installed to move water from the mining pits into the swale system. Two combinations of pumps were considered. In Option 1, the pump station would have a maximum capacity of 800 cfs. This capacity was specified by Ananta Nath (2012) from the SFWMD. This would be accomplished by installing two (2) 150 cfs and two (2) 250 cfs pumps. A second combination of pumps (Option 2) evaluated a system with a maximum capacity of 400 cfs, which consists of four (4) 100 cfs pumps. In each scenario, the pumping rate was related to the rate of flow through the diversion structure. By equating the pumping rate to the inflow rate, it is assumed that the groundwater interactions to the quarry will not change. The relationship for each option is shown in Table 9.

Table 9. Modeled Pump Station Operational Strategy

Option 1 - 800 cfs Pump Station		Option 2 - 400 cfs Pump Station	
Diversion Flow (cfs)	Pump Rate (cfs)	Diversion Flow (cfs)	Pump Rate (cfs)
0	0	0	0
0 – 150	150	0-100	100
150 – 300	300	100-200	200
300 – 550	550	200-300	300
>500	800	>300	400

- **Diversion Structure.** This structure will divert water from the Golden Gate Main Canal into the Golden Gate Quarry mining pits. In Option 1 (800 cfs pump station), this structure would consist of six (6) 6-ft wide X 4-ft high box culverts with overflow gates that would be operated based on water levels in the Golden Gate Main Canal and in the mining pit. The proposed invert elevation of the gated structures is 5.0 ft (NAVD). The gates would open if the water level in the Golden Gate Main Canal exceeds 7.93 feet (NAVD), which is consistent with the operation of the GG-3 structure. It is assumed that water levels in the canal will rise faster than water levels in the quarry and that this head differential will allow gravity flow to occur. The gates will close if the water level in the quarry is equal to or greater than that in the canal. In Option 2 (400 cfs pump station); the diversion structure would consist of four (4) gates with the same dimensions and operational strategy as in Option 1.
- **Force Main or Other Conveyance.** A force main or other conveyance facility will be required to carry the pumped water under the proposed Wilson Blvd extension that will run from the Golden Gate Canal south to I-75 where it will connect with White Lake Blvd.
- **Swale.** This feature will distribute the pumped water into the spreader systems. The swale would be approximately 10,500 feet in length and would have a bottom width of approximately 70 feet. **Figure 37** shows the typical modeled cross-section.

Figure 37. Typical Proposed Swale Cross-section

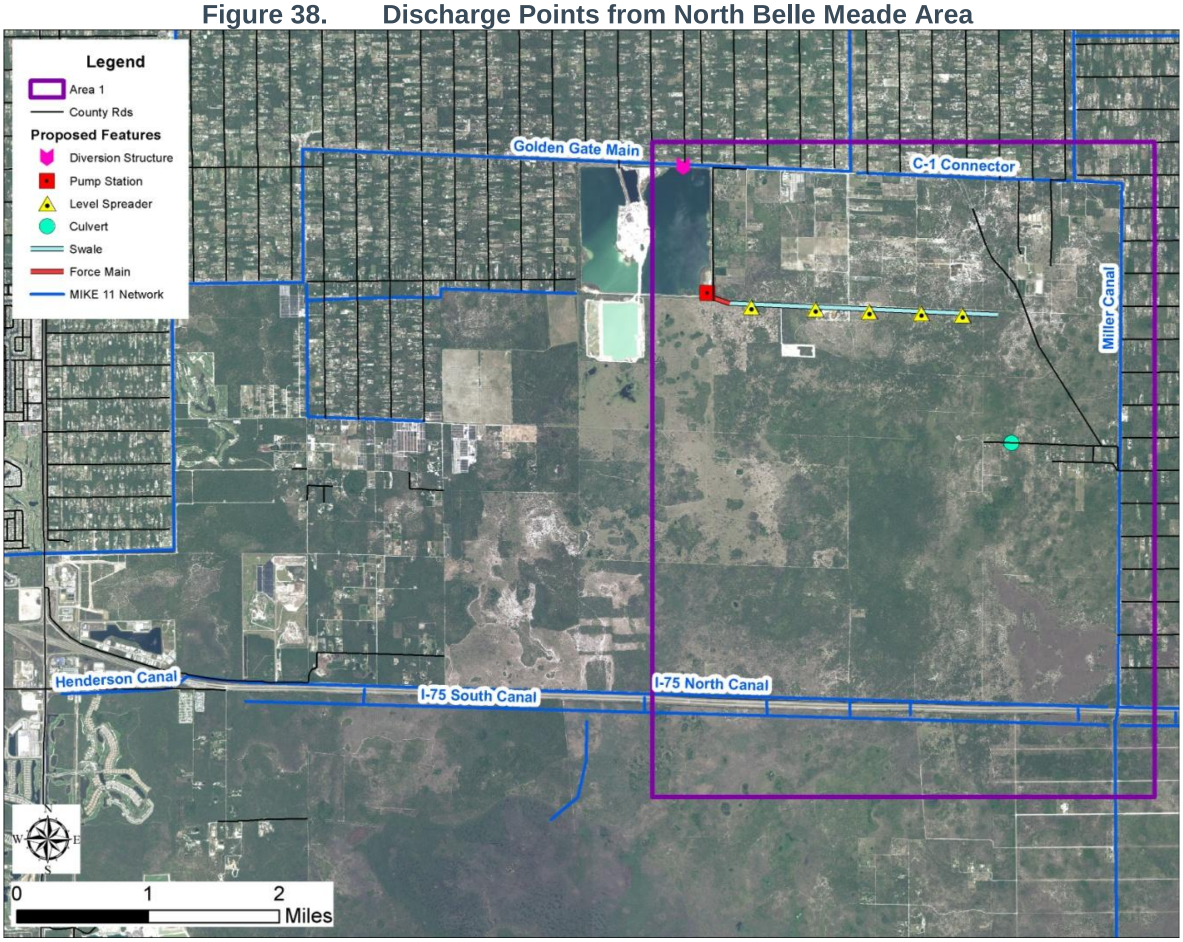


- **Level Spreaders.** It is anticipated that five (5) level spreaders would be installed on the swale to allow sheet flow from the swale to the south. Each level spreader would be 100 - 200 feet long with a minimum invert elevation of approximately 11.2 ft NAVD.

- **Culvert.** A culvert underneath Benton Rd is recommended to allow any additional overland from the spreader system to migrate to the south. It is assumed that this culvert would be 18-inches in diameter with invert elevations of approximately 8.5 feet NAVD.

In the North Belle Meade area, overland flow drains to the south and is intercepted by the I-75 North Canal as shown in **Figure 38.** From there, water will drain to the west to the Henderson Canal, to the east toward the Miller Canal, or south into the I-75 South Canal through culverts under I-75. Water in the I-75 South Canal may drain to the Miller Canal or to the south via overland flow.

Figure 38. Discharge Points from North Belle Meade Area



3.4.2. Scenario 1 – Hydrologic & Hydraulic Assessment

The calibrated MIKE SHE/MIKE 11 model of the study area was used to simulate the two proposed options for diverting water into the North Belle Meade area. The model simulations were run for the period from January 1, 2002 through October 31, 2007. The results of each option were then compared against the results of the modified existing condition model run. The following comparisons were made during this analysis.

- Pump Operation
- Hydraulic Analysis of Structures
- Flow over the new GG-3 structure
- Predicted hydroperiod in the North Belle Meade Area
- Flow to the Henderson Canal
- Flow to the Miller Canal and the Picayune Strand Restoration Area

3.4.3. Pump Operation

Figure 39 shows the timing and rate of pumping from the mine pit into the spreader system. The results indicate that zero pumping occurred in 2002 or 2007. In Option 1 (800 cfs max), the pump reaches maximum capacity only three (3) times during the course of the simulation. **Table 10** shows that in Option 1, when the pump is operating, it runs in the 150 – 300 cfs range 80 percent of the time, which is consistent with small rainfall events. The pump is predicted to operate at 550 cfs another 18 percent of the time and at full capacity only two (2) percent of the operational time.

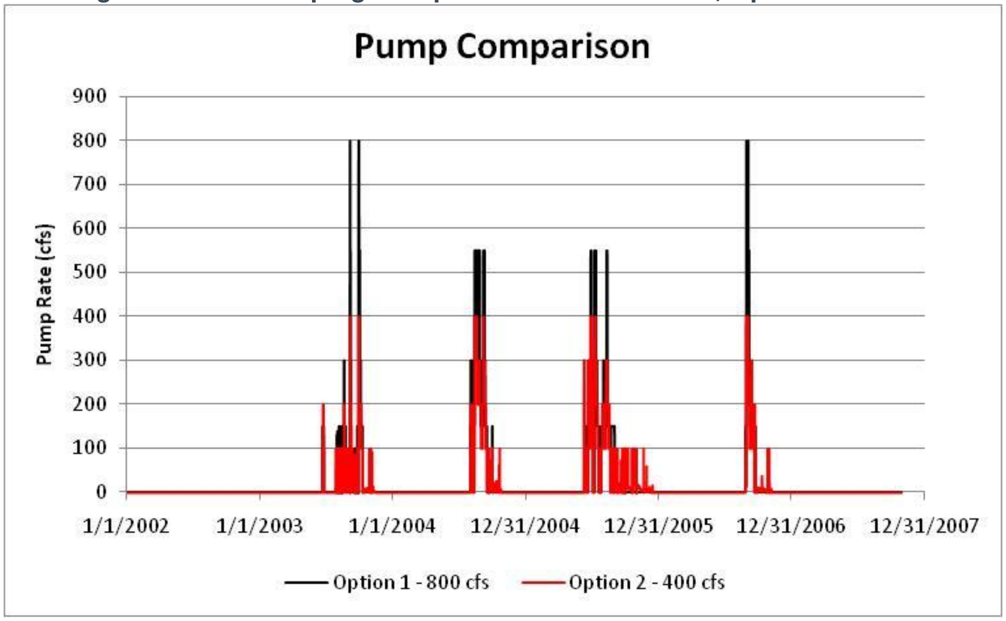
In Option 2 (400 cfs max), the pump operates at maximum capacity nine (9) percent of the operational time during the simulation (15 days) than in Option 1 (2 days). In this Option, the results indicate that the pump operates at a capacity of 200 cfs or less 78 percent of the time.

These results suggest that, from an operational perspective, a smaller pump station that is fully utilized on a regular basis may be more cost effective than a larger pump station that is rarely fully utilized.

Table 10. Percent of Time at Different Pump Rates

Option 1, 800 cfs pump			Option 2, 400 cfs pump		
Pump Rate	Days	Percent of Pumping Days	Pump Rate	Days	Percent of Pumping Days
800	3.5	2	400	15	9
550	30	18	300	24	14
300	35	21	200	55	32
150	98	59	100	79	46

Figure 39. Pumping Comparison for Scenario 1, Options 1 and 2

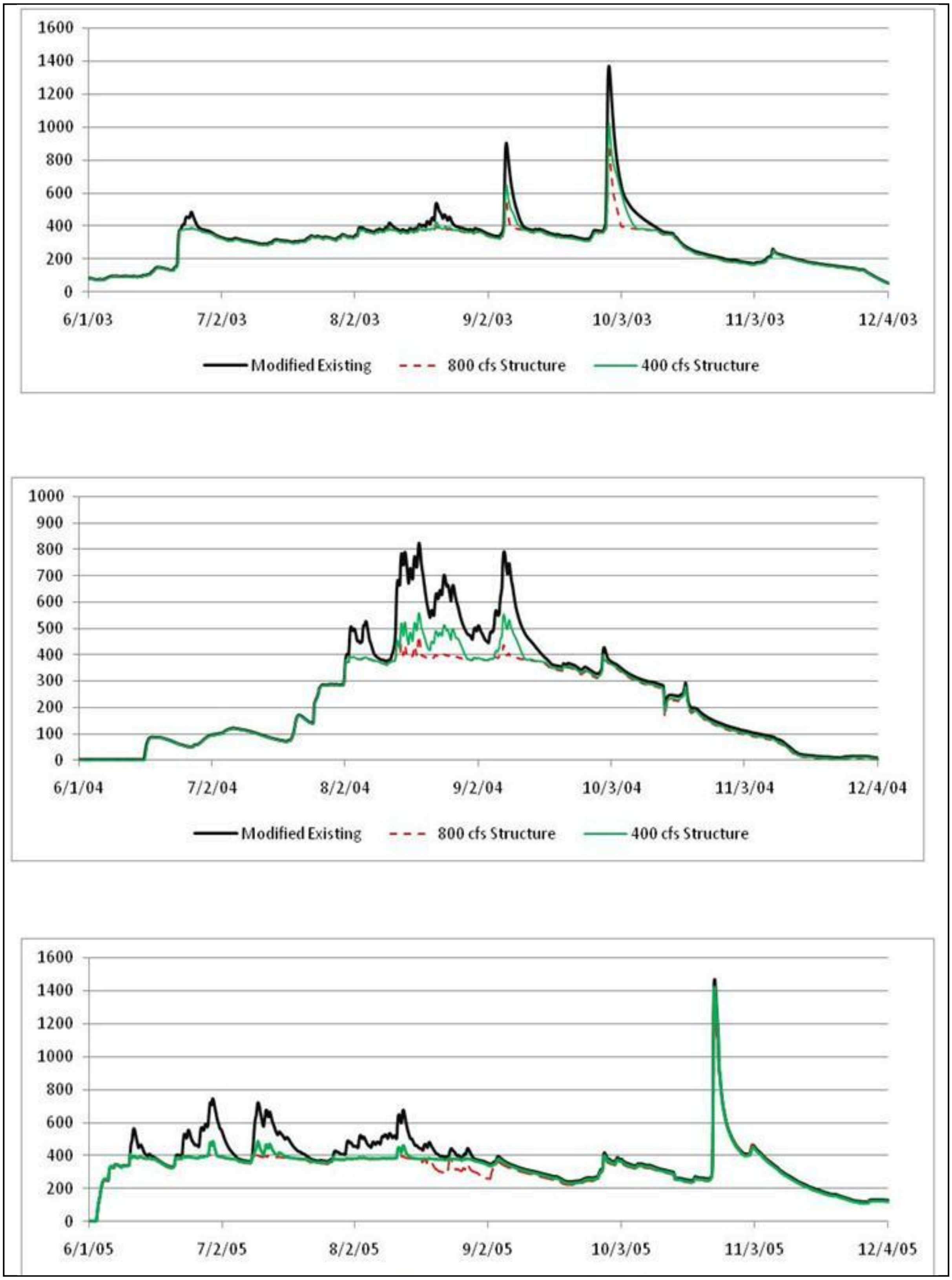


3.4.4. Flow over the GG-3 structure

Figure 40 shows a comparison of flows over the new GG-3 structure resulting from the operation of the diversion structures in Options 1 and 2. It is evident that the larger diversion structure associated with Option 1 (800 cfs pump) has a greater effect in reducing flows over the GG-3 structure during large storm events. During the storm event at the end of September 2003, Option 1 reduced the peak flow by approximately 500 cfs while Option 2 predicted a peak flow reduction of approximately 390 cfs. During the 2004, Option 1 reduced flows by 200 – 400 cfs, typically 100 cfs more than Option 2 (400 cfs pump). The reduction in peak flow over the GG-3 structure during 2005 is similar for both options suggesting that similar volumes of water were diverted out of the Golden Gate Main Canal during 2005.

In both cases, the proposed design provides the direct benefit of reducing the peak flows and the total volume of wet season flow over the GG-3 structure which may have a beneficial effect. With the maximum pump capacity of 800 cfs in Option 1, the average reduction in the wet season (July through October) discharge volume for the 2003 – 2007 water years (November – October) is predicted to equal 3,048 million gallons (**Table 11**), or approximately 13 percent. The predicted peak reduction was 5,072 million gallons during the 2004 wet season.

Figure 40. Flow Comparison over Modified GG-3 Structure



In Option 2, with a maximum pump capacity of 400 cfs the reduction in the average wet season discharge volume for 2003 – 2007 is estimated to equal 2,274 million gallons, or approximately 10 percent for the duration of the wet season. The highest wet season reduction was 3,870 million gallons 2004.

Table 11. Flow Comparison over the GG-3 structure

Average Monthly Flow Volume over GG-3 (million gallons)						
Month	Scenario 1, Option 1			Scenario 1, Option 2		
	Modified Existing	800 cfs pump	Difference	Modified Existing	400 cfs pump	Difference
January	315	282	-33	315	286	-29
February	325	319	-7	325	321	-4
March	371	367	-4	371	371	-1
April	32	31	-1	32	32	0
May	56	57	1	56	57	1
June	2390	2122	-268	2390	2137	-253
July	4226	3842	-384	4226	3887	-339
August	6887	5673	-1215	6887	5964	-924
September	7161	6102	-1059	7161	6414	-747
October	5140	4750	-389	5140	4875	-265
November	2116	2028	-88	2116	2027	-89
December	711	667	-44	711	671	-40

3.4.5. Predicted Hydroperiod in the North Belle Meade Area

Figures 41 and 42 present comparisons of the predicted change in hydroperiod relative to the Modified Existing Condition simulation. The model results indicate the resulting change in hydroperiod in the North Belle Meade area is not dependent on the pump size. Both simulations produce very similar results. In Option 1, the hydroperiod is slightly longer (5 – 10 days) in the southern and eastern portions of the North Belle Meade area. In both cases, the results indicate that when the system reaches full saturation, the combined rate of discharge from the North Belle Meade area into the Henderson Creek Canal and the Miller Canal is very similar to the rate of pumping from the mining pit.

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Figure 41. Hydroperiod Comparison:

Modified Existing Condition - Predicted Hydroperiod

Scenario 1, Option 1 – Predicted Hydroperiod

Scenario 1, Option 1 – Difference in Predicted Hydroperiod

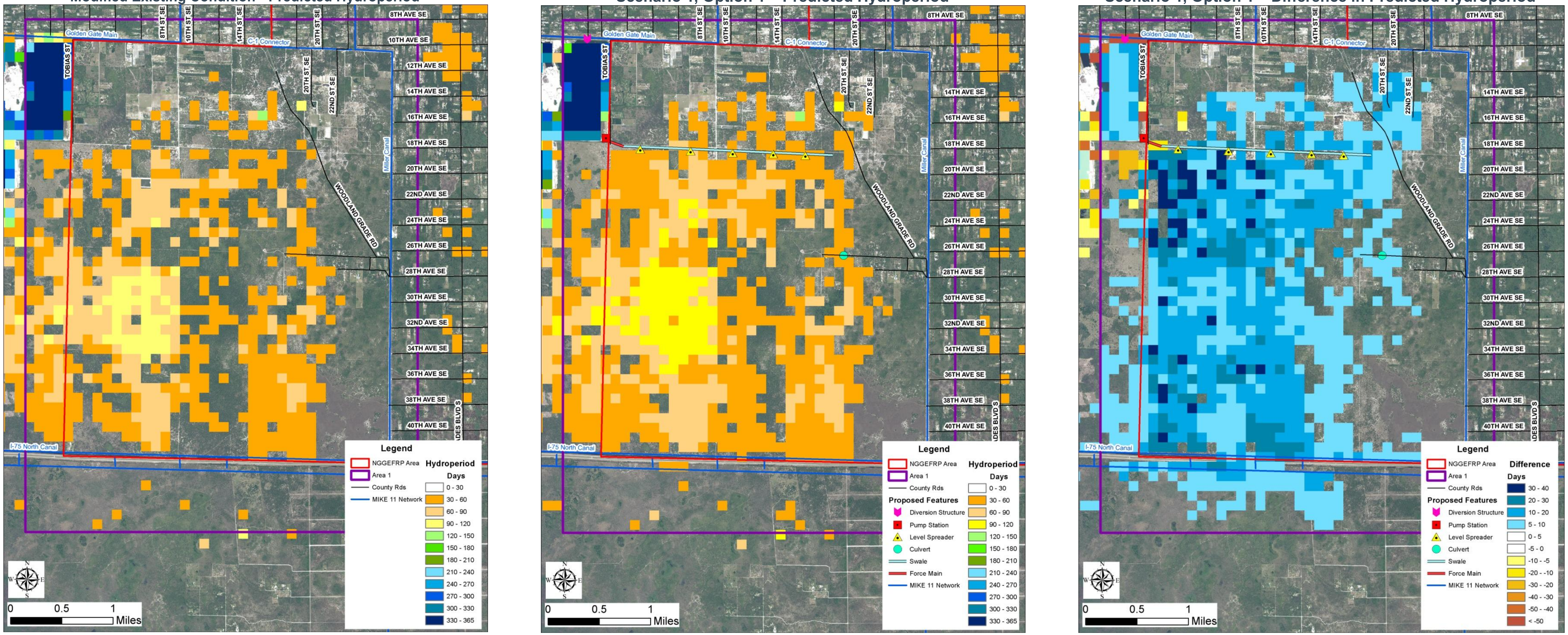
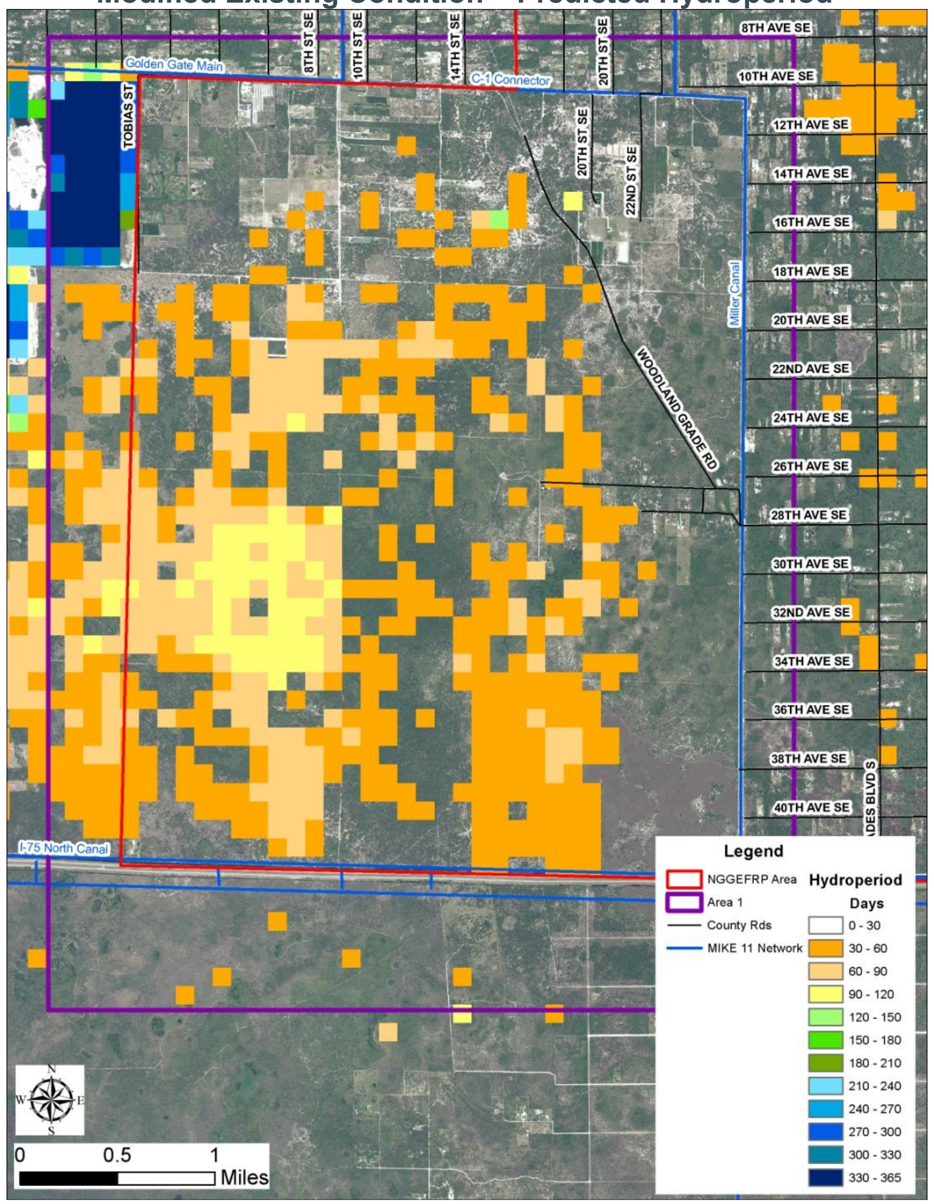
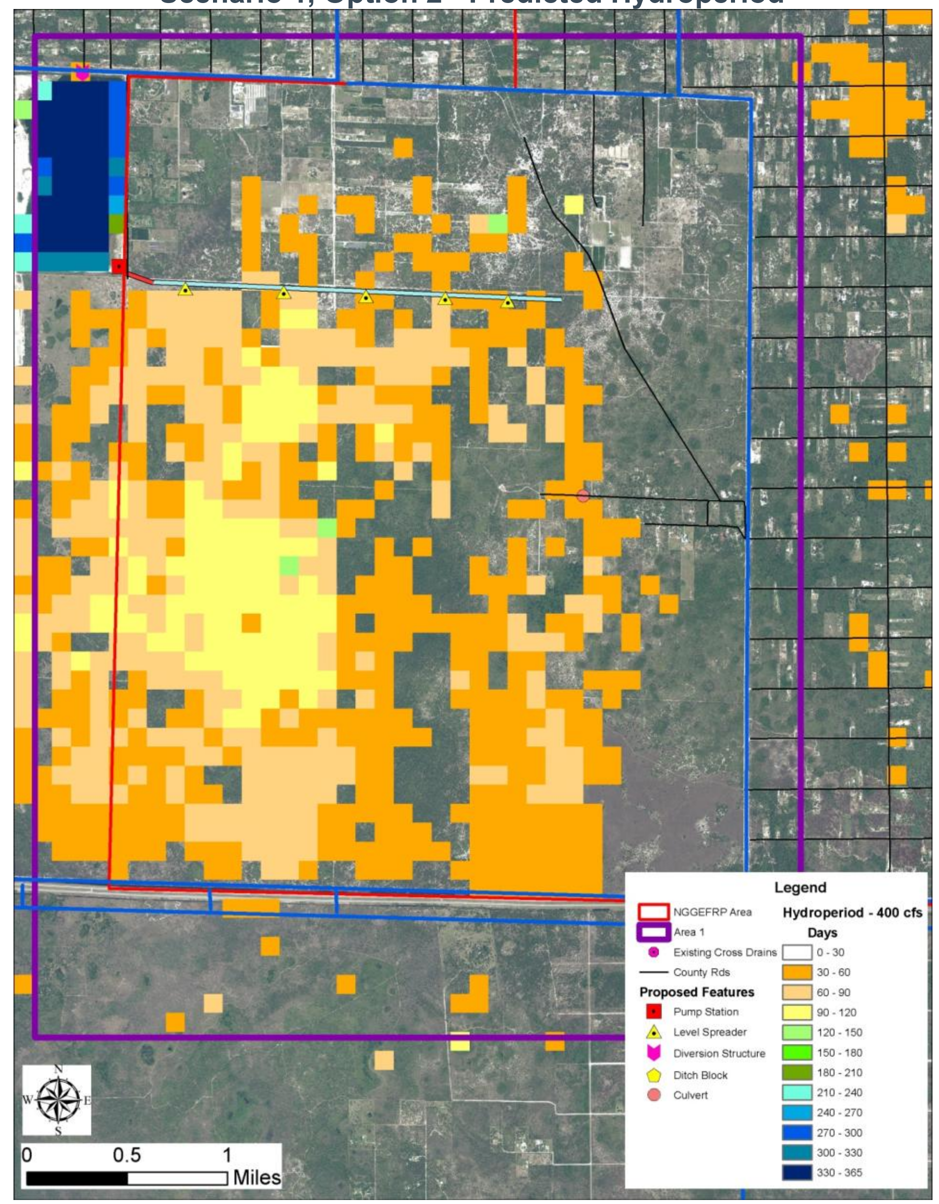


Figure 42. Hydroperiod Comparison:

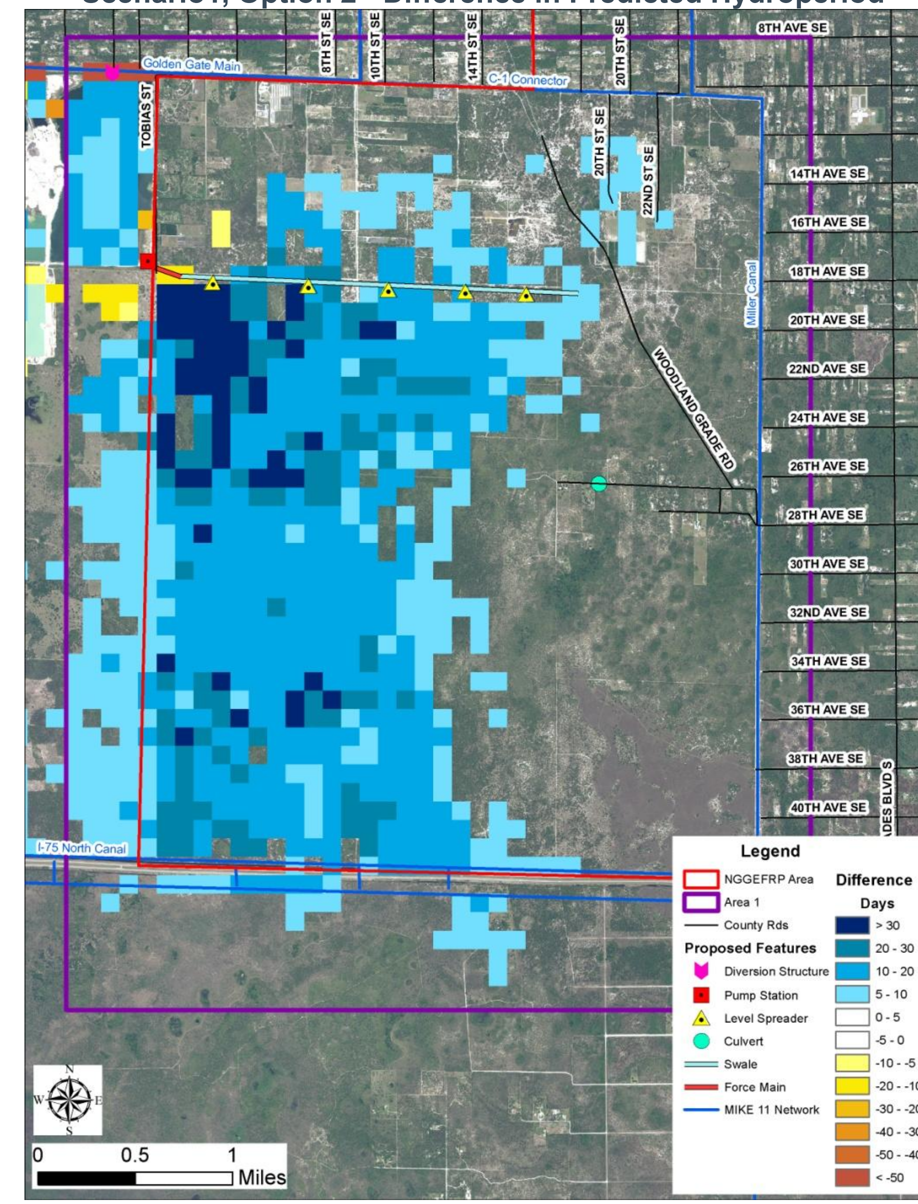
Modified Existing Condition – Predicted Hydroperiod



Scenario 1, Option 2 - Predicted Hydroperiod



Scenario1, Option 2 - Difference in Predicted Hydroperiod



This can be seen in the graphs presented in **Figures 43 and 44**. The graph in **Figure 43** shows the flows in and out of the North Belle Meade area resulting from use of the 800 cfs pump station. **Figure 44** shows similar graphs associated with the use of the 400 cfs pump station.

Figure 43. Combined Flow Comparison
Scenario 1, Option 1 – 800 cfs Pump Station

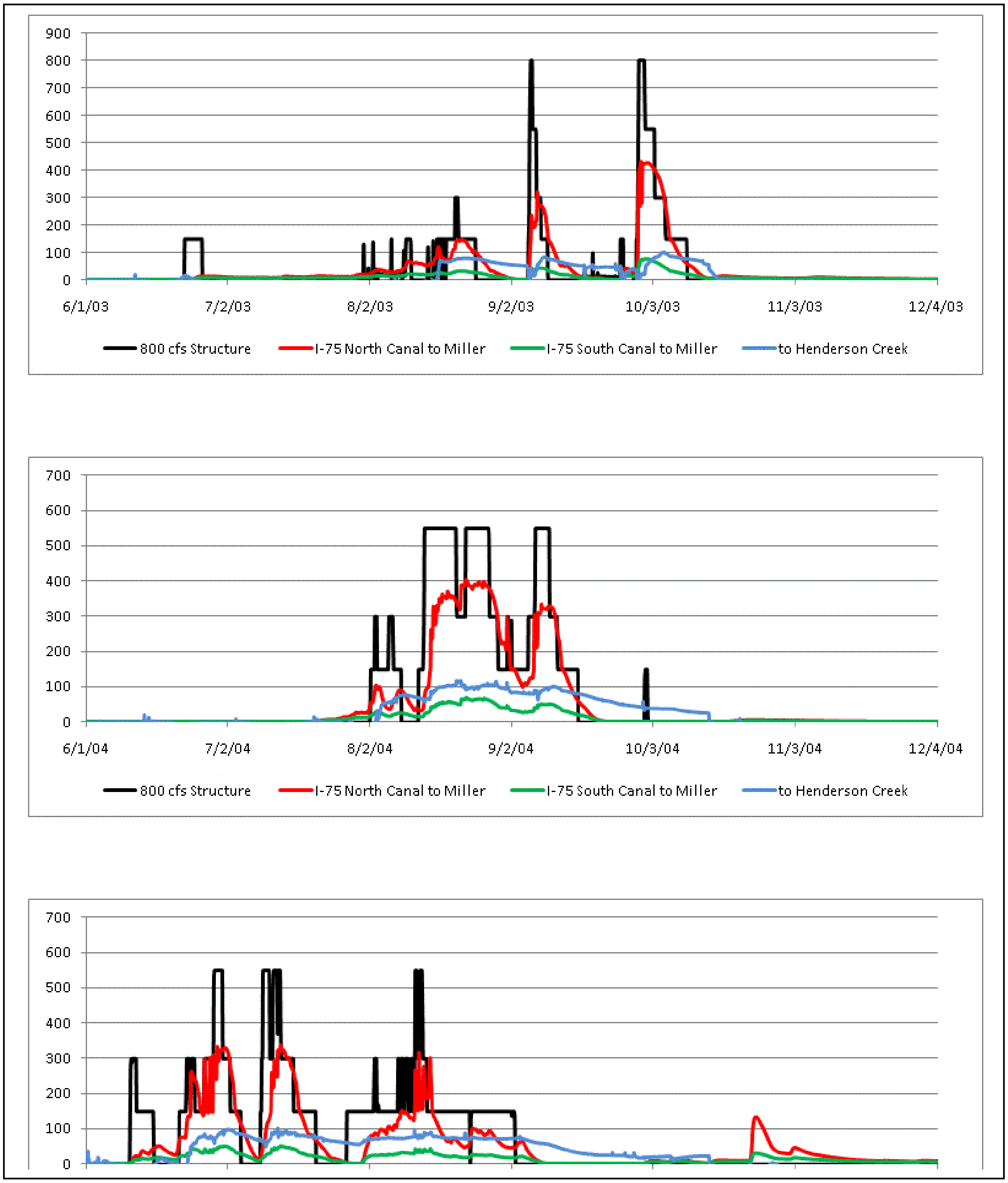
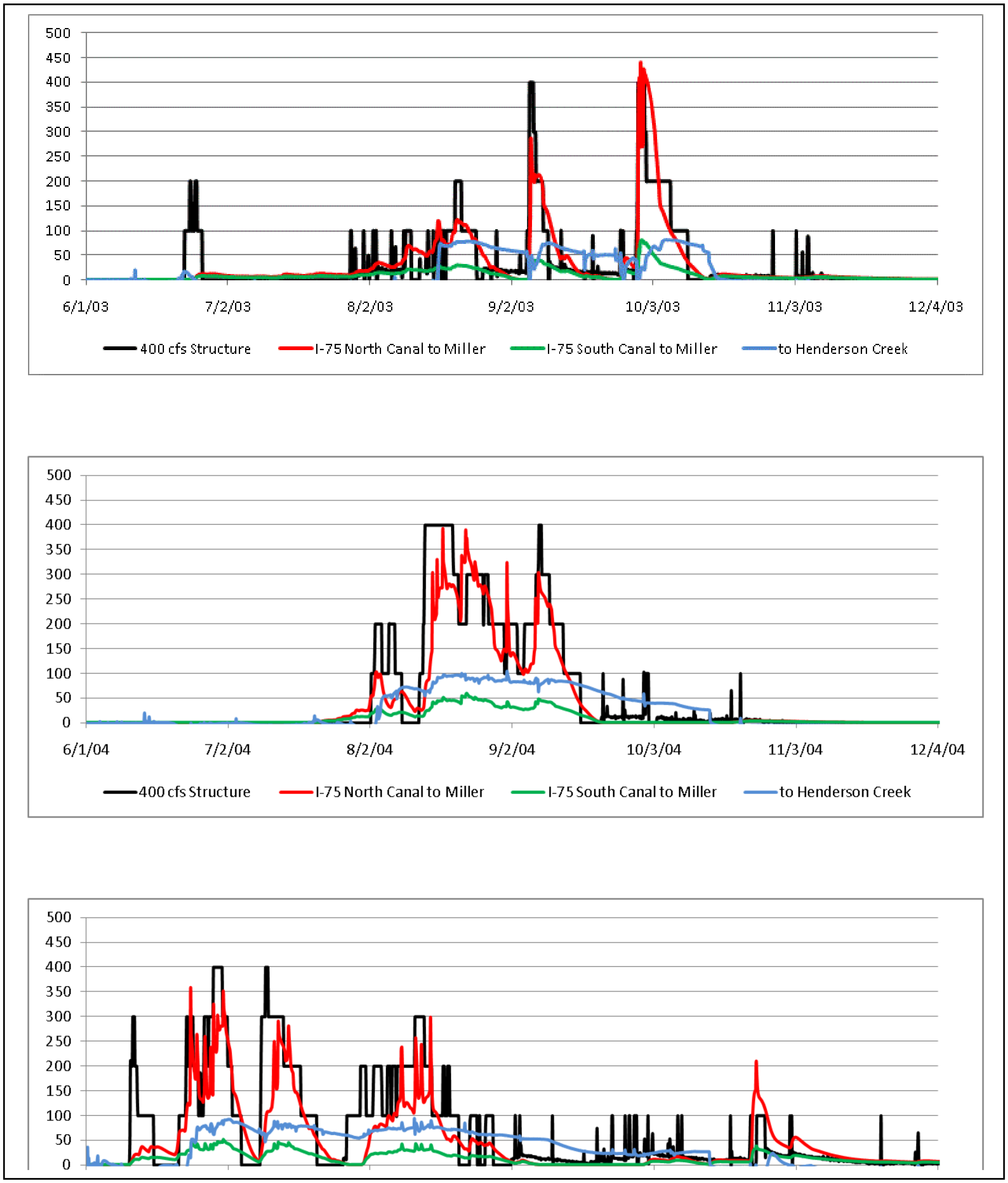


Figure 44. Combined Flow Comparison
Scenario 1, Option 2 – 400 cfs Pump Station



In both options, the system becomes fully saturated after 2 – 3 weeks of pumping from the mining pit and the total flows leaving the North Belle Meade area are very similar to the inflows. In Option 1, the diversion pump only operated at the maximum capacity of 800 cfs for a total of three (3) days during the simulation period. During those events, the outflow rate from North Belle Meade did not match the inflow rates; however, the outflow hydrographs indicate high flows for longer periods of time. This is consistent with the slightly longer average hydroperiod results predicted near I-75.

3.4.6. Flow Comparison to Henderson Creek

Figure 45 and **Table 12** present comparisons of the predicted change in flow to Henderson Creek for the Option 1(800 cfs pump) and Option 2 (400 cfs pump) scenarios. In both cases, there is an increase in flow of 40 percent or more to Henderson Creek during in the 2003 – 2006 wet seasons. In Option 1, the predicted increase in flow to Henderson Creek during the 2004 and 2005 wet seasons reaches a peak rate that is approximately double the flow predicted by the modified existing conditions model. The increased volume equals approximately 204 million gallons per month from July through October. The peak increase in predicted flow volume to Henderson Creek is 1,428 million gallons during the 2005 wet season.

In Option 2 (400 cfs pump station), the predicted peak flows are more than one-third that predicted by the modified existing conditions model. The predicted average increase in monthly volume is 172 million gallons for the 2003 – 2006 wet seasons, with a peak increase of 1,229 million gallons during the 2005 wet season.

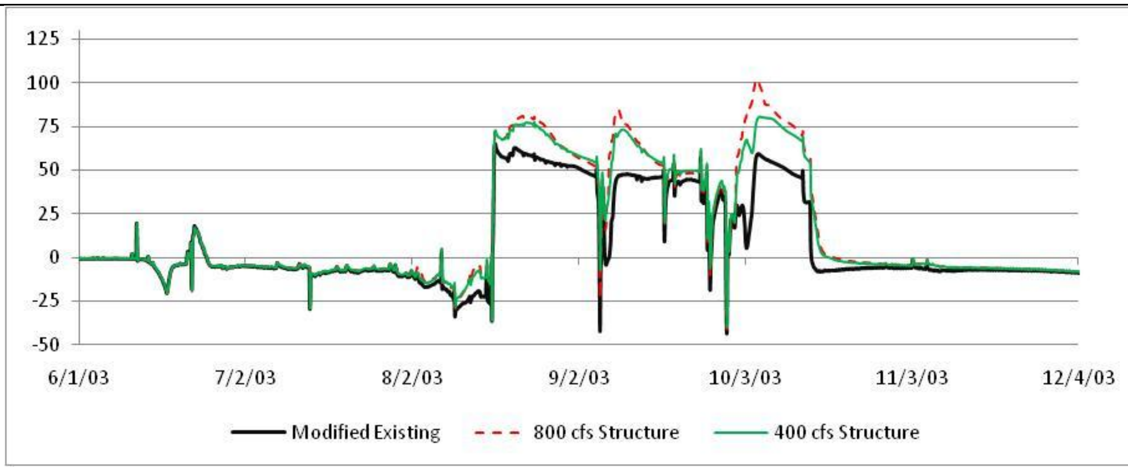
The Collier County Watershed Management Plan (Atkins 2011) concluded that Rookery Bay receives too much water for the months of June – October, and too little water during the remainder of the year. The results of the North Belle Meade diversion indicate that the predicted increase in flow to Henderson Creek may increase the flow surplus from July through August. However, it may contribute to reducing the identified flow deficit in October and November.

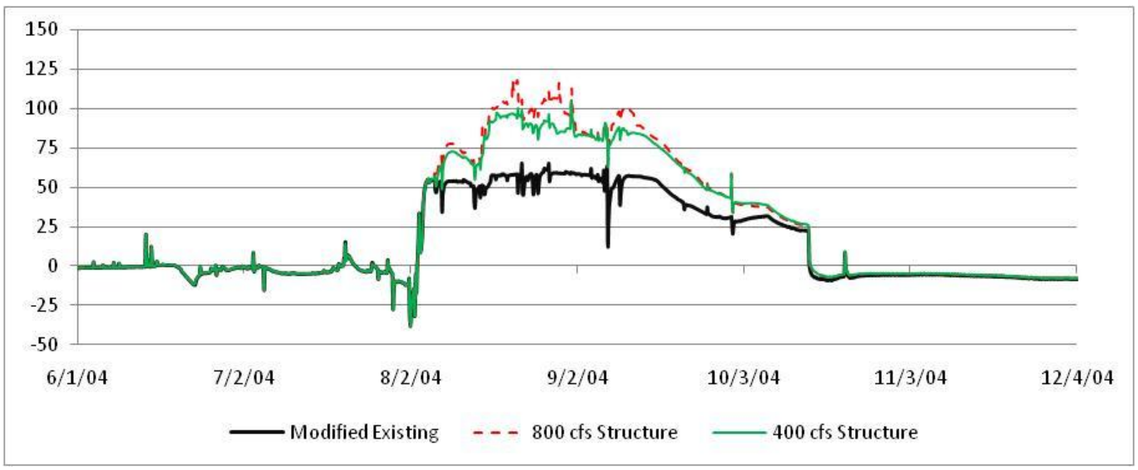
Table 12. Flow Comparison to Henderson Creek

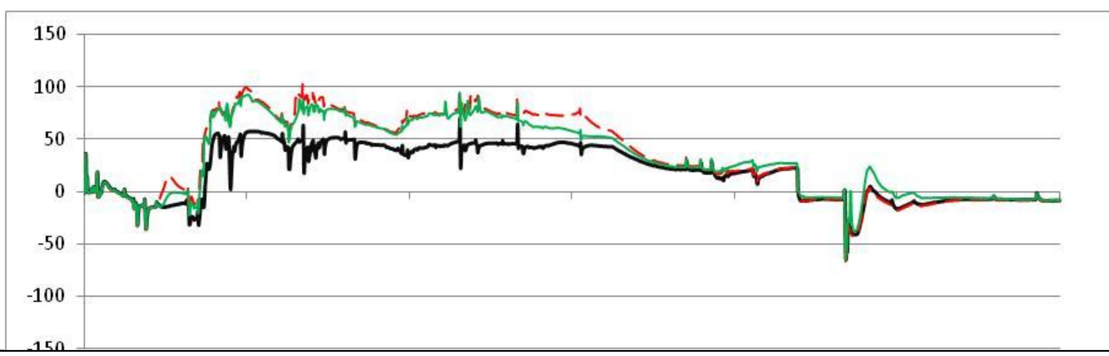
Average Monthly Flow Volume into Henderson Creek (million gallons)						
Month	Scenario 1, Option 1			Scenario 1, Option 2		
	Modified Existing	800 cfs pump	Difference	Modified Existing	400 cfs pump	Difference
January	-206	-205	1	-206	-202	4
February	-176	-176	-1	-176	-173	3
March	-187	-187	0	-187	-186	2
April	-138	-139	0	-138	-138	1
May	-35	-35	0	-35	-35	0
June	-3	51	54	-3	38	42
July	181	300	119	181	283	102
August	425	710	286	425	654	229
September	648	957	310	648	892	244
October	149	253	104	149	261	113
November	-127	-121	6	-127	-105	22
December	-165	-162	2	-165	-159	5

In both options, the results suggest that additional measures would be required to store or divert the increased flow volume into Henderson Creek during the months of June through September in order to minimize any potential effects to the Rookery Bay Estuary. The CCWMP recommended off-line storage from the Henderson Canal and a spreader swale in the western Picayune Strand State Forest south of I-75. These projects may be able to offset the increase in wet season flows.

Figure 45. Flow Comparison to Henderson Creek







3.4.7. Flow Comparison to Miller Canal and the Picayune Strand Restoration Area.

The Picayune Strand Restoration Project (PSRP) includes construction of new pump stations on the Miller and Faka Union Canals that will move water from the canal network into spreader swales. This section discusses the change is stage in the Miller Canal upstream of the PSRP pump station, and the predicted change in the pump rate for the new Miller Canal pump station.

Graphs that present a flow comparison from the I-75 North Canal into the Miller Canal during 2003, 2004, and 2005 are shown in **Figure 46**. **Table 13** presents a comparison of the flow volume from the I-75 North Canal into Miller Canal. As with the rate of flow to the Henderson Creek Canal; the rate of flow to the Miller Canal is higher and of longer duration in Option 1 (800 cfs pump) than in Option 2 (400 cfs pump).

In Option 1, the volume of water discharging into the Miller Canal during August and September is almost triple the volume predicted by the modified existing conditions model. In Option 2, the increased volume approaches 2.5 times the predicted volume of the modified existing conditions model. The increased flow to the Miller Canal **(Figure 46)** has the effect of increasing the stage in the canal, although it does not appear to contribute to an increased hydroperiod in portions of the Golden Gate Estates east of the Miller Canal.

Table 13. Flow Comparison from I-75 North Canal to Miller Canal

Average Monthly Flow Volume to Miller Canal (million gallons)						
Month	800 cfs pump			400 cfs pump		
	Modified Existing	Scenario 1	Difference	Modified Existing	Scenario 1	Difference
January	1	1	0	1	1	0
February	2	2	0	2	2	0
March	1	1	0	1	1	0
April	0	0	0	0	0	0
May	0	0	0	0	0	0
June	143	298	156	143	273	130
July	148	521	373	148	427	279
August	576	1702	1127	576	1384	808
September	535	1530	995	535	1211	675
October	257	467	210	257	390	133
November	83	83	1	83	92	10
December	17	17	0	17	17	0

Figure 46. Flow Comparison from I-75 North Canal to Miller Canal

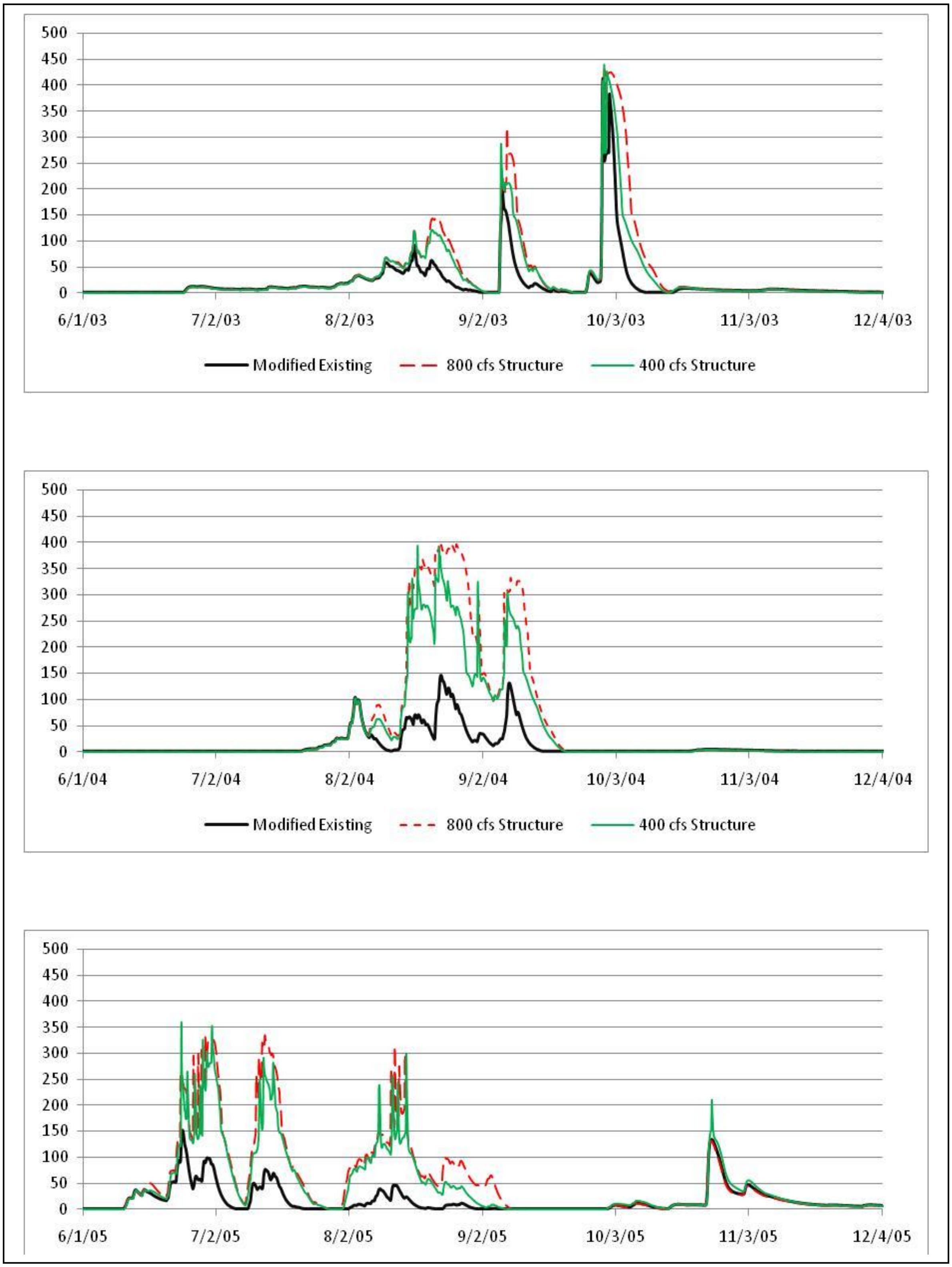


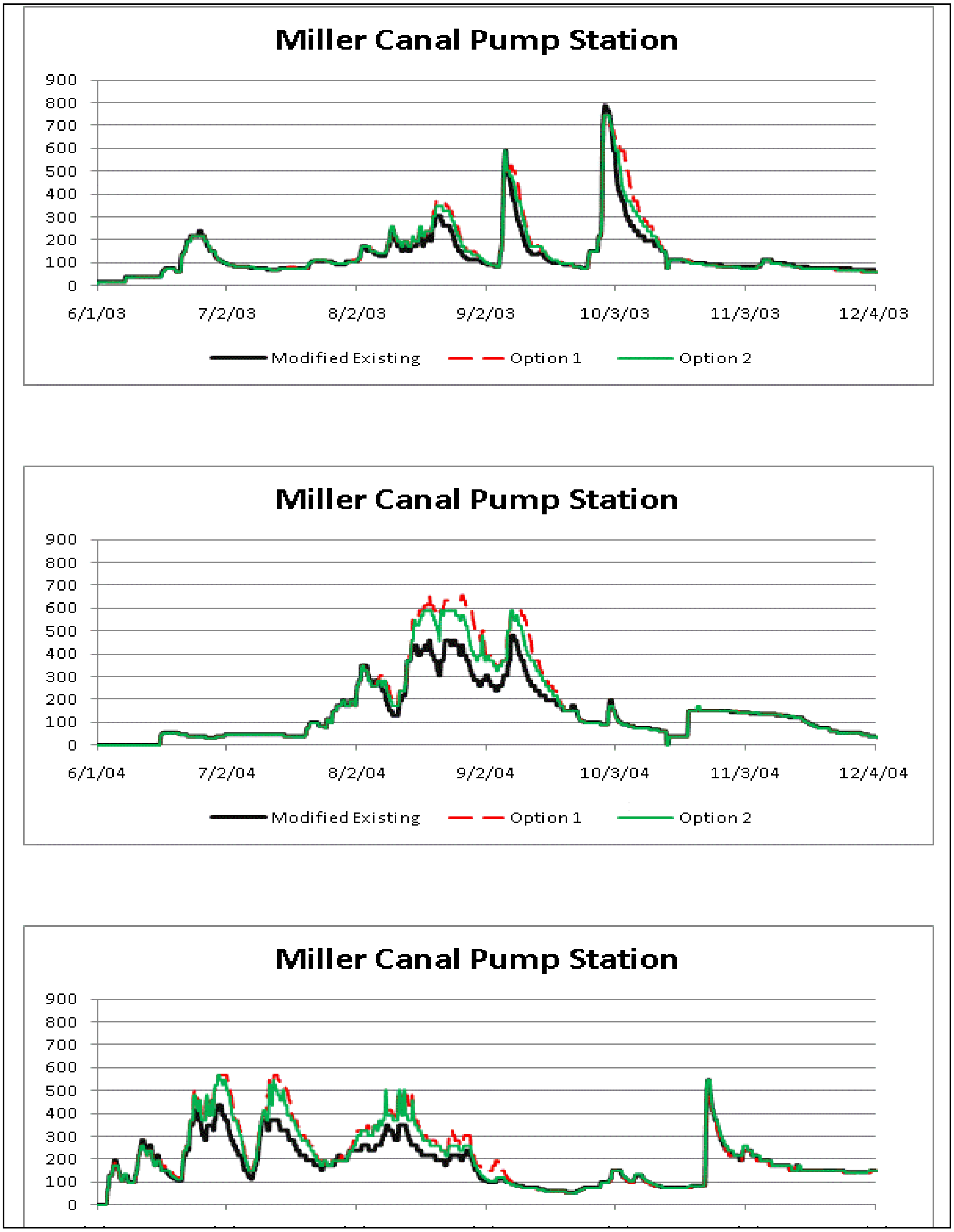
Table 14 presents a comparison of the pumped volumes of water into the PSRP area through the new Miller Canal pump station. The increase in wet season pumping is direct result of the increased discharge to the Miller Canal from the North Belle Meade project. The pumping rates through the proposed Miller Canal pump station are predicted to increase by as much as 150 cfs (**Figure 47**) during the wet season in both cases; however the pump rate associated with Option 1(800 cfs diversion) reaches a higher peak level and extends for a longer duration.

The increase in the volume flow in Miller Canal as a result of implementation of Scenario1, Option 1 is insignificant, and does not appear to adversely impact the conveyance capacity of the canal, nor of the roadside swales that drain to the canal. Do to the limited size of the model domain, it is unclear how the PSRP area and the Ten Thousand Islands estuary would be affected by the additional volume of water pumped from the Miller Canal; however, the maximum predicted pump rate of 800 cfs through the new Miller Canal pump station remains well under the maximum capacity of 1250 cfs. This indicates that this project will neither impact the targeted delivery of water to the pumps, nor the overall goals of the PSRP, adversely.

Table 14. Comparison of Miller Canal Pump Station Flow Volume to the Picayune Strand Restoration Area

Average Monthly Flow Volume Through Miller Pump Station (million gallons)						
Month	Scenario 1, Option 1			Scenario 1, Option 2		
	Modified Existing	800 cfs pump	Difference	Modified Existing	400 cfs pump	Difference
January	442	445	3	442	454	12
February	172	171	-1	172	172	1
March	128	128	0	128	128	1
April	22	22	0	22	22	0
May	7	7	0	7	7	0
June	1196	1307	111	1196	1287	92
July	1908	2264	356	1908	2172	264
August	3736	4690	953	3736	4428	691
September	3495	4355	860	3495	4083	589
October	2211	2381	171	2211	2325	114
November	1626	1614	-12	1626	1627	0
December	1057	1037	-20	1057	1046	-11

Figure 47. Flow Comparison to the Picayune Strand Restoration Area



3.4.8. Scenario 1 – Conclusions

The following conclusions were drawn from the analysis completed for Area 1.

- Outflows to Naples Bay may be reduced by an average of three (3) percent per month during the wet season if an 800 cfs pump station is utilized. Outflows may be reduced by 2.5 percent per wet season month with a 400 cfs pump station.
- The North Belle Meade area has limited storage and infiltration capacity such that outflows closely match inflows after only a few weeks of operation.
- Using an 800 cfs pump station may double the predicted flows to Henderson Creek and could increase the existing flow surplus to Rookery Bay during the period from June – September unless other projects are implemented to divert or store the additional flow. A 400 cfs pump may increase flows to Henderson Creek by more than one-third.
- The increased flows to Henderson Creek would likely contribute to minimization of the flow deficit to the Rookery Bay Estuary during the months of October and November.
- The pumping rates in the proposed Miller Canal Pump station are increased, but remain well below the maximum designed capacity indicating the there is no effect on delivery of water to the pumps or on the overall goals of the PSRP.

The results of the North Belle Meade Spreader Swale project analysis suggest that the use of a diversion system that features an 800 cfs pump station is viable if additional projects are implemented to mitigate the increased flows, and minimize or prevent an increase in flow to the Rookery Bay Estuary. However, the 800 cfs pump station configuration considered in this evaluation may not be cost effective since the pump rarely operates at maximum capacity.

3.5. Scenario 2 – Wetland Connectivity North of Oil Well Road

3.5.1. Description of Scenario

The Area 2 Scenario considers the interconnection of the wetland systems north of Oil Well Rd between the Golden Gate and Faka Union Canals. Many cross-drains have been installed in this area, with the primary intent of reducing road inundation during rain events. The existing cross drains are shown in **Figure 48**.

The proposed method to connect the wetland features is to install additional culverts in appropriate locations in the area. These locations were selected through the evaluation of the topography and wetland areas completed in Sub-Tasks 2.1 and 2.2. The proposed culvert locations are shown in **Figure 49** and described in **Table 15**. Invert elevations were estimated from a road elevation taken from the Digital Elevation Model (DEM) at each proposed culvert location. It is assumed that each culvert would have 18 inches of cover, so invert elevations were estimated to be 36 – 42 inches below the road elevation.

Figure 48. Scenario 2 – Existing Cross Drains

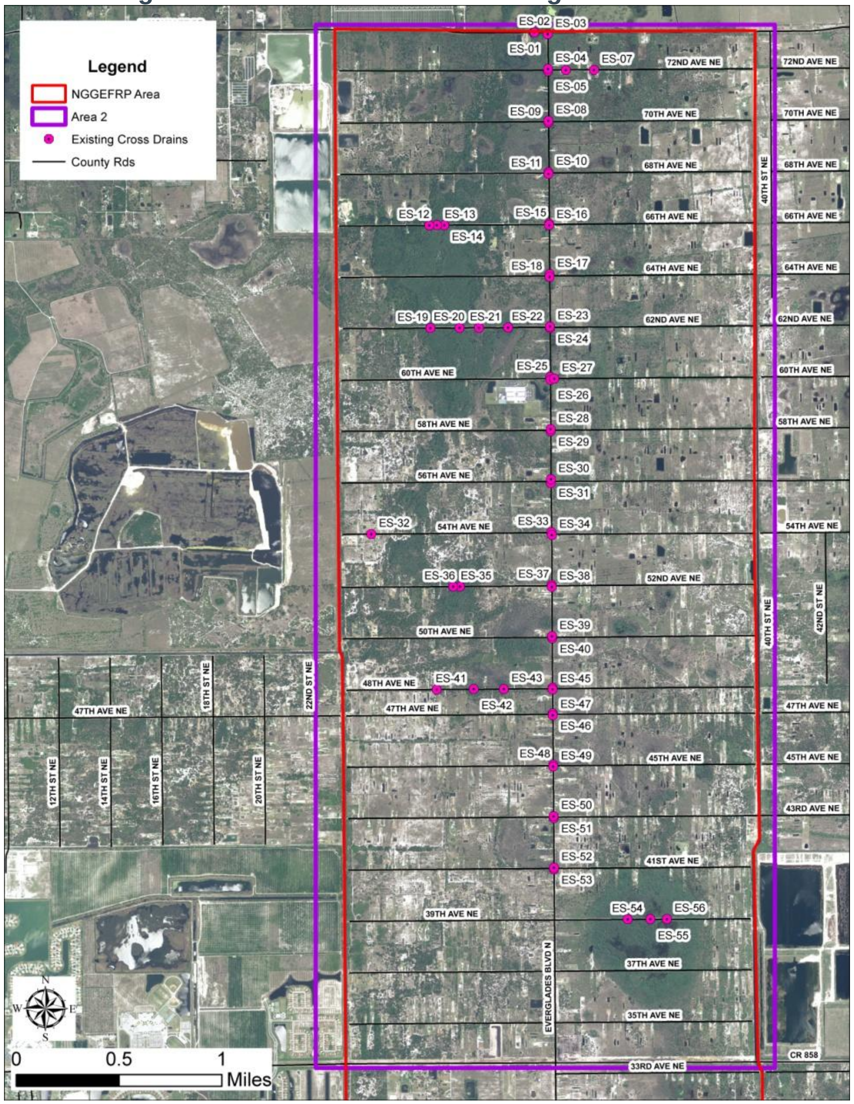


Figure 49. Scenario 2 – Proposed Cross Drains

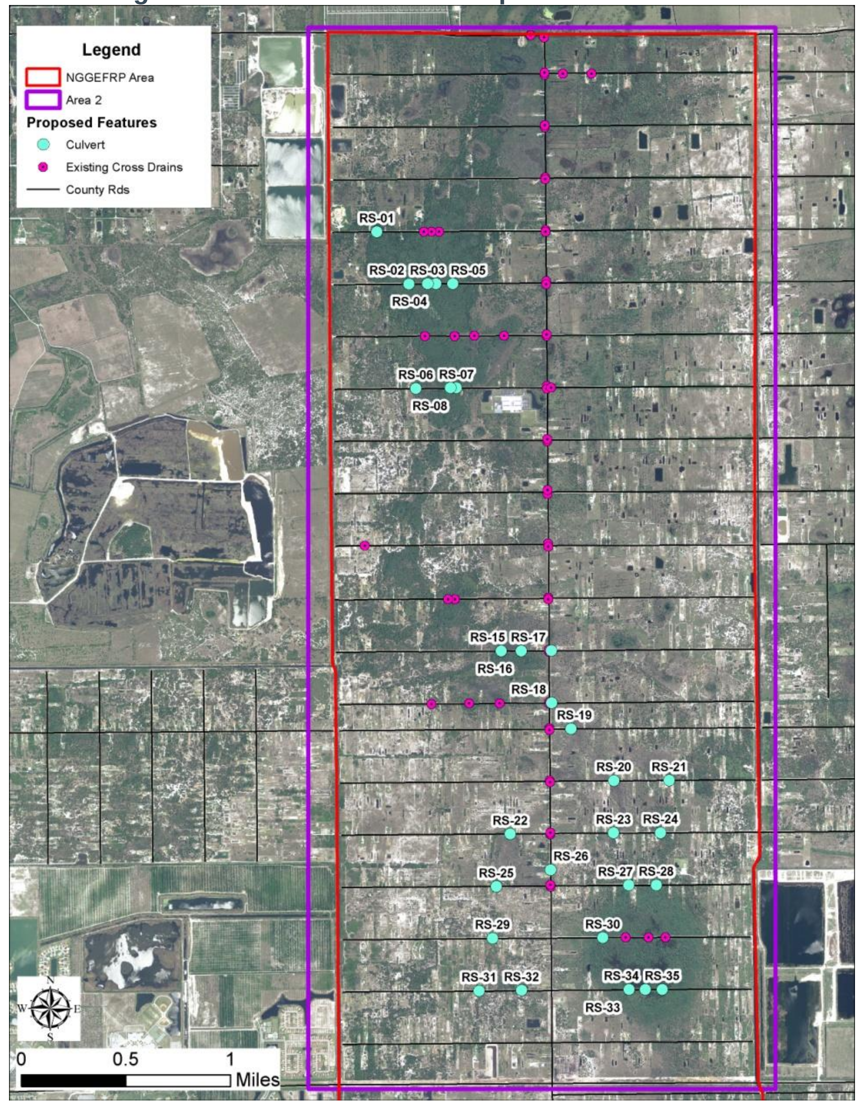


Table 15. Proposed Additional Culverts in the Scenario 2 Area.

Recommended Structures							
Structure ID	Description	X Coordinate	Y Coordinate	Upstream Invert	Downstream Invert	Diameter (in)	Length (ft)
RS-01	Culvert	473035	734181	15.92	15.90	18	40
RS-02	Culvert	473847	732863	14.85	14.83	24	40
RS-03	Culvert	474317	732867	14.85	14.83	24	40
RS-04	Culvert	474526	732871	14.85	14.83	24	40
RS-05	Culvert	474958	732869	14.85	14.83	24	40
RS-06	Culvert	474021	730227	15.60	15.58	24	40
RS-07	Culvert	474898	730237	15.60	15.58	24	40
RS-15	Culvert	476179	723603	15.08	15.06	18	40
RS-16	Culvert	476682	723603	15.08	15.06	18	40
RS-17	Culvert	477441	723608	15.22	15.20	18	60
RS-18	Culvert	477450	722289	15.21	15.19	18	60
RS-19	Culvert	477939	721639	14.64	14.62	18	40
RS-20	Culvert	479026	720327	14.66	14.64	18	40
RS-21	Culvert	480422	720333	15.00	14.98	24	40
RS-22	Culvert	476401	718981	14.73	14.71	18	40
RS-23	Culvert	479019	719004	14.64	14.62	18	40
RS-24	Culvert	480203	719010	14.57	14.55	24	40
RS-25	Culvert	476057	717655	14.50	14.48	24	40
RS-26	Culvert	477429	718078	14.53	14.51	18	50
RS-27	Culvert	479397	717690	12.75	12.73	24	40
RS-28	Culvert	480098	717694	12.75	12.73	24	40
RS-29	Culvert	475959	716346	13.51	13.49	24	40
RS-30	Culvert	478746	716363	11.31	11.29	24	40
RS-31	Culvert	475623	715020	13.21	13.19	18	40
RS-32	Culvert	476694	715034	13.21	13.19	18	40
RS-33	Culvert	479406	715050	12.28	12.26	24	40
RS-34	Culvert	479822	715055	12.28	12.26	24	40
RS-35	Culvert	480246	715057	12.28	12.26	24	40

3.5.2. Scenario 2 – Hydrologic & Hydraulic Assessment

This section summarizes the results of the simulations completed to evaluate the effects of installing the cross-drains defined in **Table 15.** This section discusses the results for each of the following issues:

- Flow in the Golden Gate and Faka Union Canals at Oil Well Rd.
- Predicted Change in Hydroperiod
- Predicted Change in Seasonal Average and High Water Levels

3.5.3. Flow in the Golden Gate and Faka Union Canals at Oil Well Rd

Table 16 presents a comparison of the average monthly flow volume for the 2003 – 2007 water years in the Golden Gate and Faka Union Canals at Oil Well Rd. These locations are at the southern end of the Scenario 2 Area. Flow hydrographs for these locations are shown in **Figures 50 and 51.** The results indicate that the addition of the new culverts contributes to a reduction in the peak flow rates in the receiving canals.

The results for the Golden Gate Canal shown in **Table 16** indicate that there is an overall annual decrease in the flow volume within the Golden Gate Canal. This is characterized by less flow (a negative difference) throughout the year. The slight increases in flow during July and December are likely associated with increased baseflow.

These results also show a change in the flow pattern in the Faka Union Canal. The peak flow rate is lower; however the hydrograph extends for a longer period of time before returning to baseflow conditions. One example of the modified flow pattern is evident in the period from September 1, 2003 through October 15, 2003 in the Faka Union Canal. This result suggests that more water is stored in the wetland systems and discharges to the Faka Union Canal over a longer period of time.

Table 16. Scenario 2 Flow Volume Comparison

Average Monthly Flow Volume (million gallons)						
Month	Golden Gate Canal at Oil Well Rd.			Faka Union Canal at Oil Well Rd.		
	Modified Existing	Scenario 2	Difference	Modified Existing	Scenario 2	Difference
January	640	640	0	61	60	-1
February	461	461	0	45	44	-1
March	458	458	0	44	43	0
April	205	204	0	18	18	0
May	98	98	0	33	33	-1
June	1555	1542	-13	266	266	0
July	2732	2738	6	397	390	-7
August	4221	4212	-9	653	632	-21
September	4462	4462	0	638	620	-18
October	3404	3401	-3	353	347	-5
November	2044	2039	-5	155	149	-7
December	1154	1156	2	94	90	-4

Figure 50. Flow Comparison - Golden Gate Main Canal at Oil Well Rd

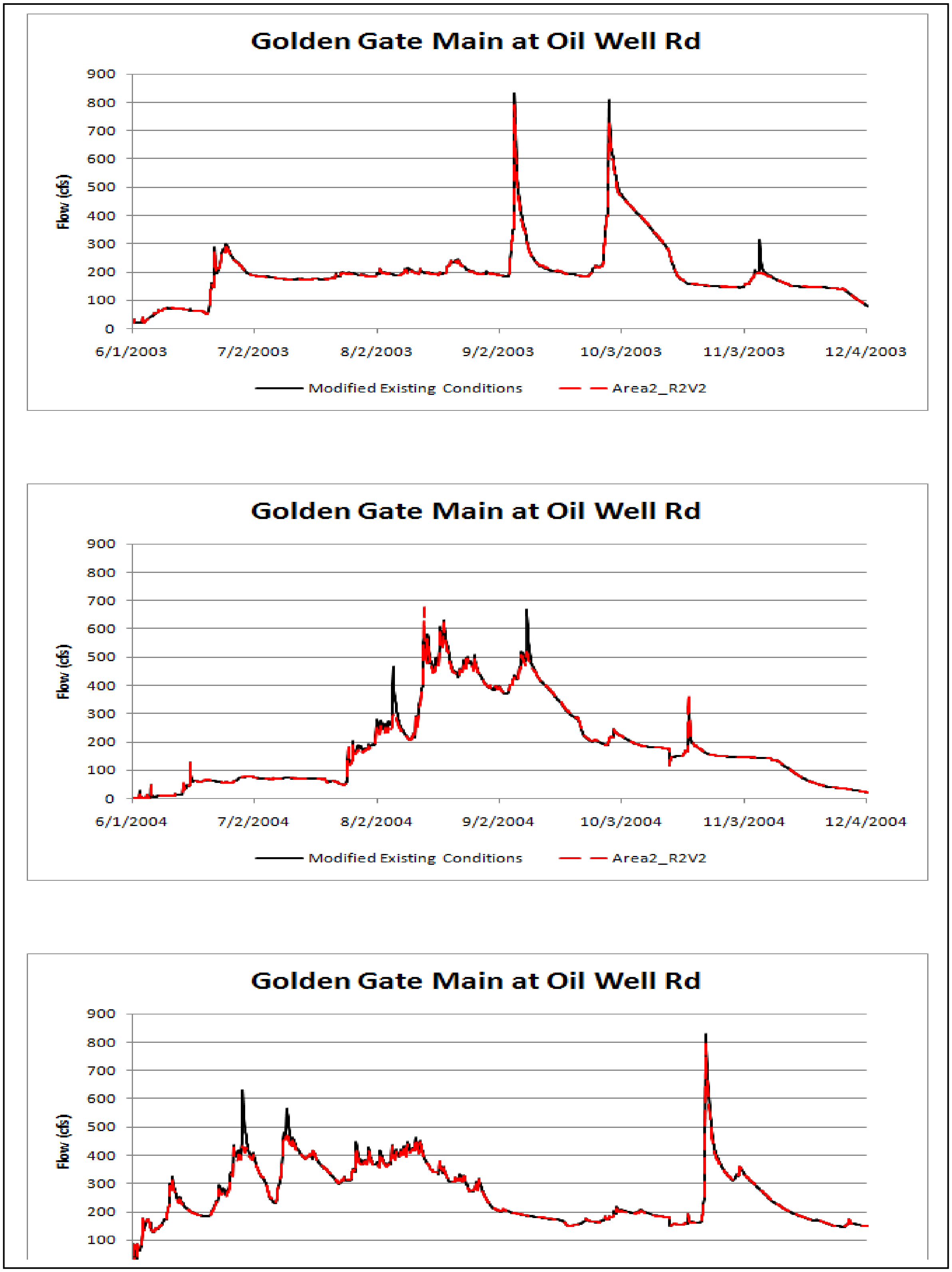
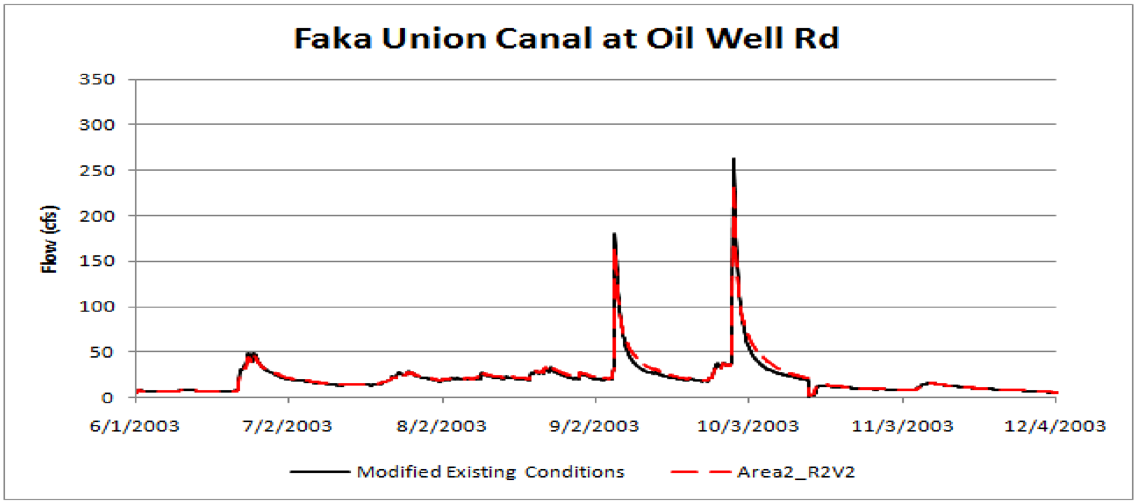
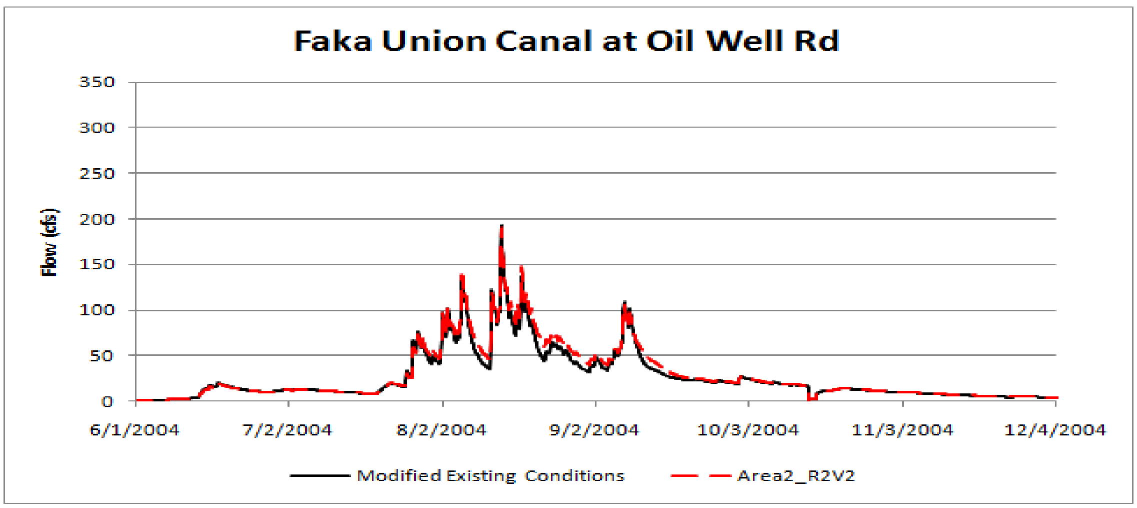
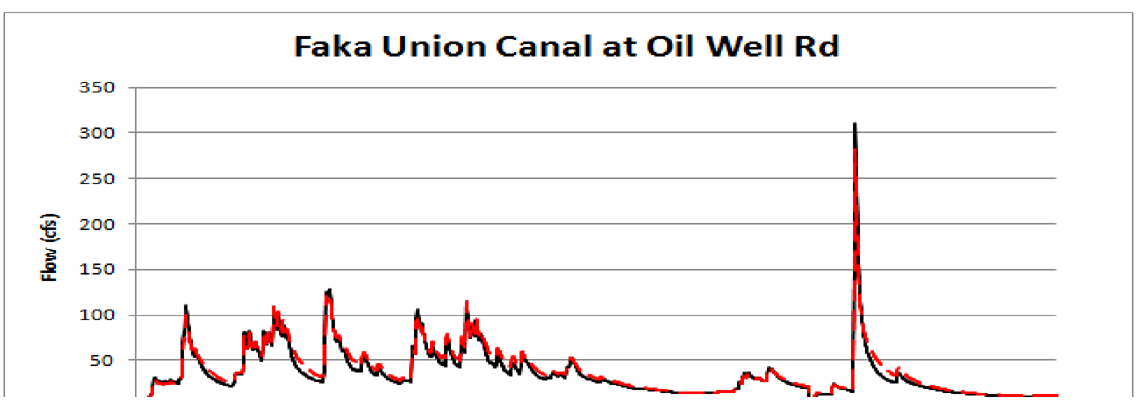


Figure 51. Flow Comparison - Faka Union Canal at Oil Well Rd







3.5.4. Predicted Hydroperiod in the Scenario 2 Area

Figure 52 presents a comparison of the hydroperiod results from the Modified Existing Condition and the hydroperiod results of the Scenario 2 simulations. The comparison shows that construction of the additional culverts may result in the hydroperiod being reduced by as much of 50 days in the area west of Everglades Blvd, between 66th Ave NE and 56th Ave NE. This area is commonly known as Panther Walk. There does not appear to be a concurrent increase in hydroperiod south of Panther Walk.

These results suggest that the current lack of culverts tends to trap water in the Panther Walk wetland system by the lack of culverts under 60th and 64th Avenues NE. It appears that adding these culverts allows water to flow south from Panther Walk and rapidly infiltrate to groundwater.

The comparison also shows a slight increase in hydroperiod of 5 – 10 days in the Winchester Head area, between 41st Ave NE and 35th Ave NE, east of Everglades Blvd. The increased hydroperiod in Winchester Head corresponds with a predicted decrease in hydroperiod in the areas immediately north of Winchester Head from 47th Ave NE to 41st Ave NE.

3.5.5. Predicted Change in Seasonal Average and High Wet Levels

Figure 53 shows the difference in the predicted average wet season groundwater level. **Figure 54** shows the difference in the predicted seasonal high groundwater level. Positive values indicate that the new cross-drains have contributed to an increase in the groundwater elevation. Negative values suggest that the cross-drains have contributed to a decrease in the groundwater elevation.

Both of these graphs present results that are consistent with the predicted changes in the hydroperiod. It appears that lack of cross-drains may actually be beneficial to the wetland systems in some instances. One example is the Panther Walk area where the addition of culverts may allow more water to drain from the existing cypress wetland system resulting in an average groundwater elevation that is 2 – 3 inches lower. Similar results are seen in the area north of Winchester Head, where the average groundwater elevation is predicted to drop by 1 – 2 inches.

Figure 52. Hydroperiod Comparison

Modified Existing - Predicted Hydroperiod

Scenario 2 - Predicted Hydroperiod

Difference in Predicted Hydroperiod

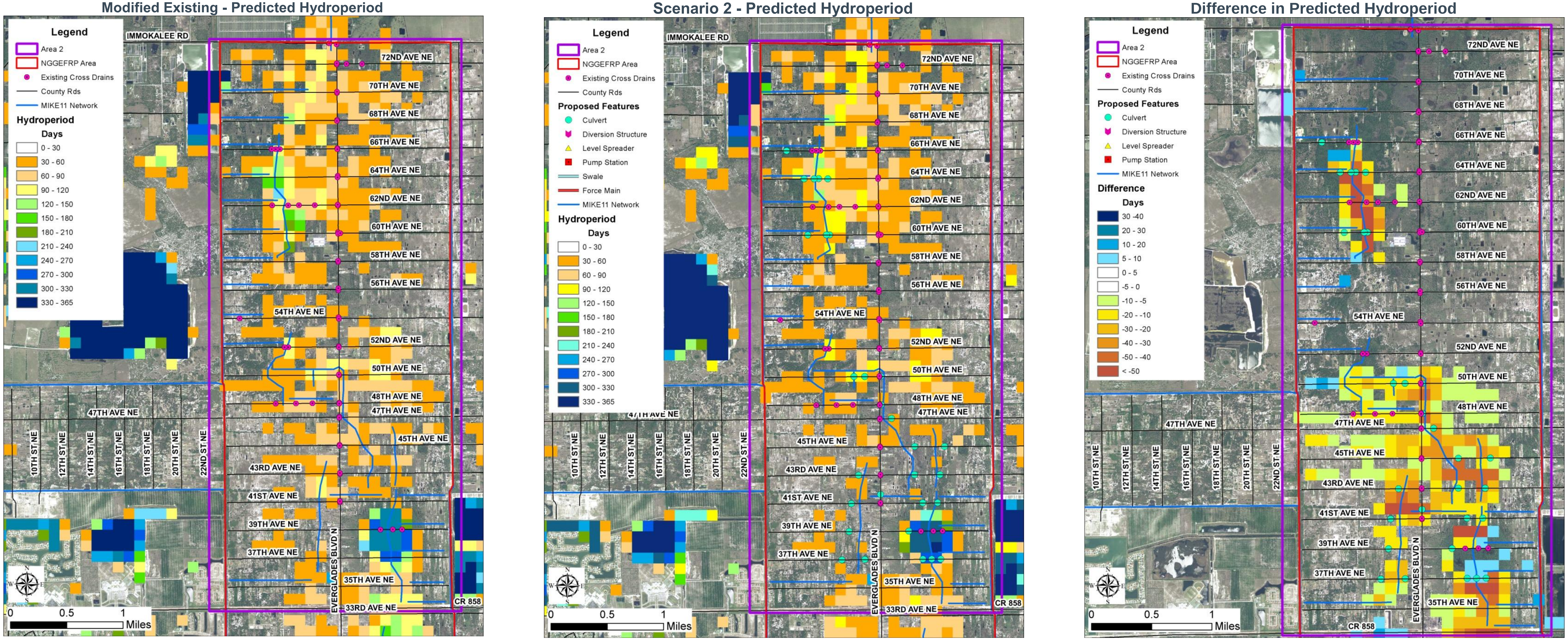


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Figure 53. Scenario 2 – Difference in Predicted Wet Season Average Groundwater Elevation

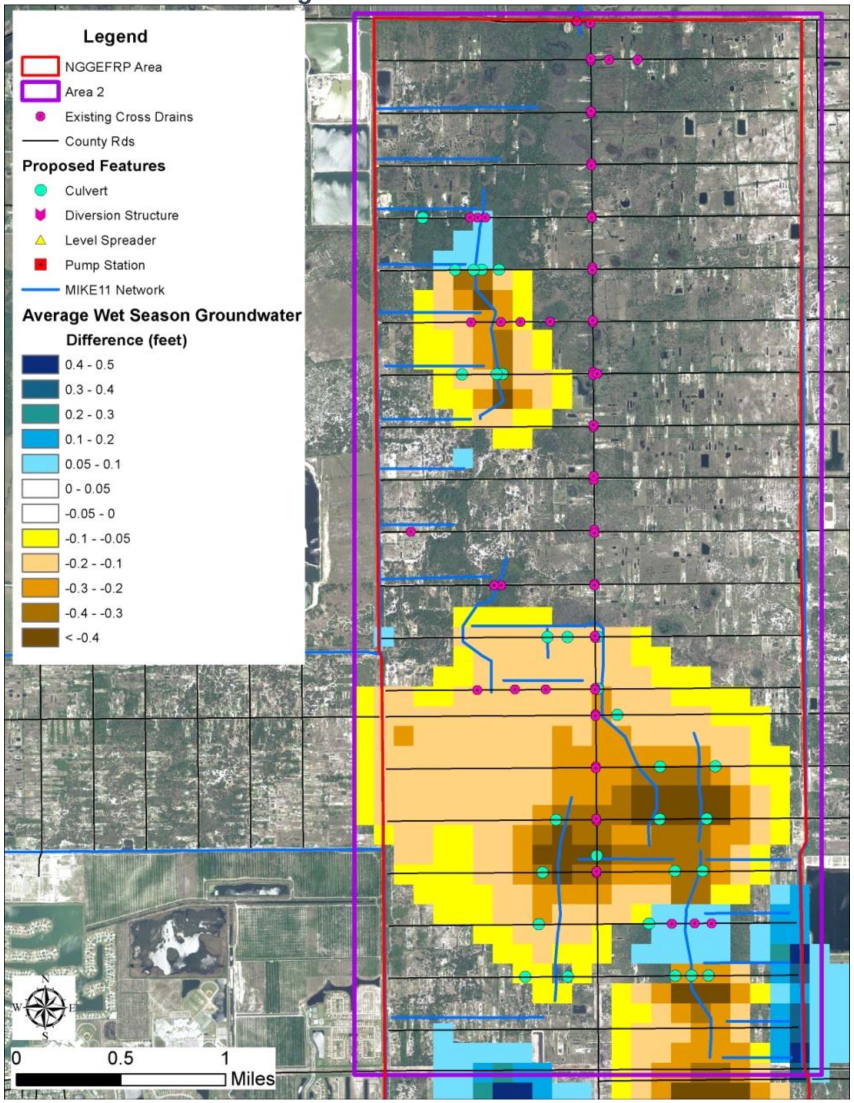
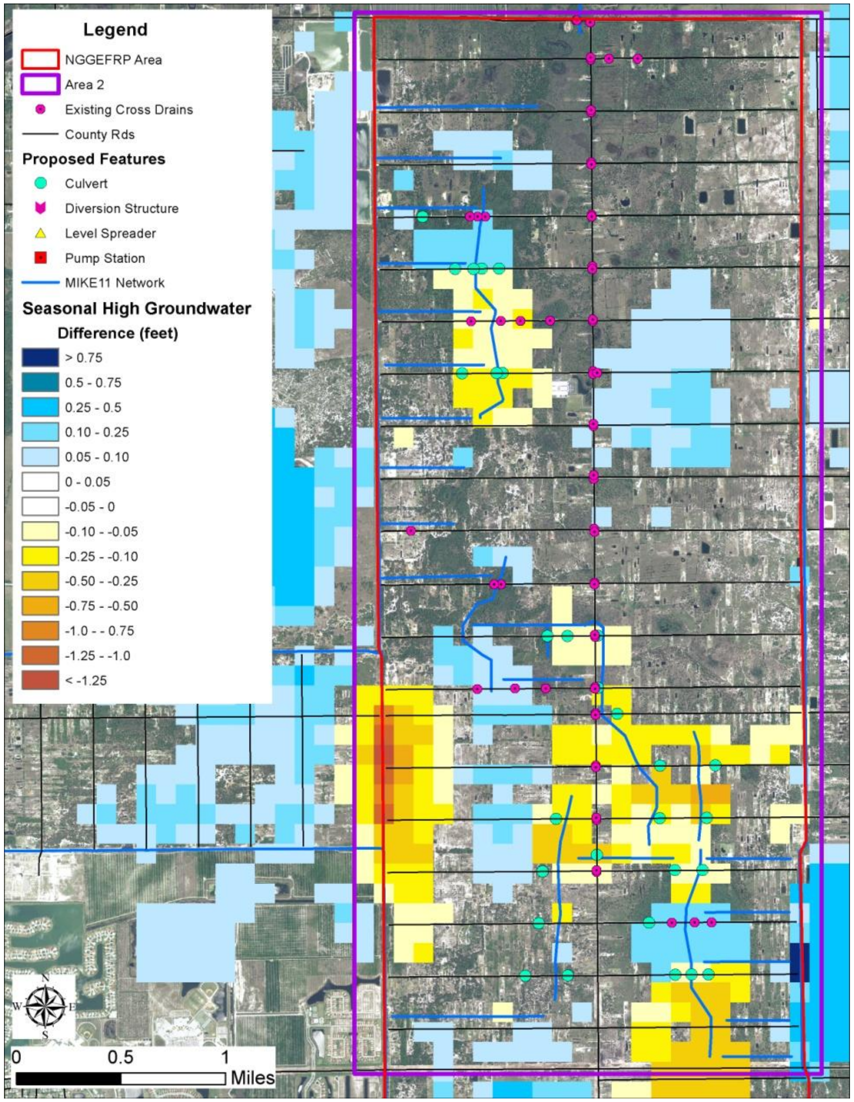


Figure 54. Scenario 2 – Difference in Predicted Seasonal High Groundwater Elevation



3.5.6. Scenario 2 – Conclusions

The following conclusions were drawn from the analysis completed for Scenario2.

- The addition of the proposed cross-drains contributes to increased infiltration and contributes to a slight reduction in predicted flows in the Golden Gate and Faka Union Canals.
- Model results suggest that the addition of the proposed cross-drains may have the unintended consequence of draining existing wetland areas.

The results of the Scenario 2 analysis suggest that adding cross-drains in the areas around the Panther Walk and Winchester Head wetland areas may be detrimental to these preserved wetland systems. Collier County should use caution when deciding how to proceed in the area north of Oil Well Rd.

3.6. Scenario 3 – Wetland Connectivity South of Oil Well Rd.

3.6.1. Description of Scenario

The Area 3 Scenario considers the interconnection of the wetland systems south of Oil Well Rd to I-75 between the Golden Gate/Miller Canals and the Faka Union Canal. This scenario also evaluates the potential benefit of cross-drains in the area between Jung Blvd E and Golden Gate Blvd east of 8th St NE. Several cross-drains have been installed in this area, with the primary intent of reducing road inundation during rain events. The existing cross drains are shown in **Figure 55**.

The proposed method to connect the wetland features is to install additional cross-drains in appropriate locations in the area. As in Scenario 2, proposed cross-drain locations were determined through evaluation of wetland areas and the DEM. The proposed cross-drain locations considered in this analysis are shown in **Figure 56** and described in **Table 17**.

Table 17. Proposed Culverts Included in the Scenario 3 Analysis

Recommended Structures							
Structure ID	Description	X Coordinate	Y Coordinate	Upstream Invert (ft)	Downstream Invert (ft)	Diameter (in)	Length (ft)
RS-36	Culvert	479469	711095	12.19	12.17	24	40
RS-37	Culvert	479928	709793	12.15	12.13	24	40
RS-38	Culvert	478138	708447	11.64	11.62	24	40
RS-39	Culvert	480051	708472	12.03	12.01	24	40
RS-40	Culvert	478177	707129	10.53	10.51	24	40
RS-41	Culvert	477073	705760	11.61	11.59	24	40
RS-42	Culvert	477760	705762	11.08	11.06	24	40
RS-43	Culvert	473739	704412	11.05	11.03	24	40
RS-44	Culvert	476047	704427	11.20	11.18	24	40
RS-45	Culvert	477584	704445	11.40	11.38	24	60
RS-46	Culvert	477897	704446	11.40	11.38	24	40
RS-47	Culvert	473747	703089	11.10	11.08	24	40
RS-48	Culvert	474915	703090	11.70	11.68	24	40
RS-49	Culvert	477598	703122	11.11	11.09	24	60
RS-50	Culvert	477747	703123	11.11	11.09	24	40
RS-51	Culvert	473931	701768	11.40	11.38	24	40
RS-52	Culvert	478930	701823	11.82	11.80	24	40

Recommended Structures							
Structure ID	Description	X Coordinate	Y Coordinate	Upstream Invert (ft)	Downstream Invert (ft)	Diameter (in)	Length (ft)
RS-53	Culvert	467871	701062	11.00	10.98	24	60
RS-54	Culvert	467868	700401	10.50	10.48	24	60
RS-55	Culvert	473563	700449	11.00	10.98	24	40
RS-56	Culvert	480145	700505	11.20	11.18	24	40
RS-57	Culvert	468284	699085	10.20	10.18	24	40
RS-58	Culvert	473520	699129	11.00	10.98	24	40
RS-59	Culvert	466504	697752	9.90	9.88	24	40
RS-60	Culvert	467874	697787	10.00	9.98	24	60
RS-61	Culvert	469924	697779	18.90	18.88	24	40
RS-62	Culvert	473911	697813	11.00	10.98	24	40
RS-63	Culvert	474415	697814	11.00	10.98	24	40
RS-64	Culvert	473763	696493	10.50	10.48	24	40
RS-65	Culvert	477632	696524	10.70	10.68	24	60
RS-66	Culvert	474576	695137	10.20	10.18	24	40
RS-67	Culvert	467900	694889	9.50	9.48	24	40
RS-68	Culvert	467898	694759	9.50	9.48	24	40
RS-69	Culvert	469226	693554	10.20	10.18	24	40
RS-70	Culvert	478877	693855	11.00	10.98	24	40
RS-71	Culvert	479582	692541	10.70	10.68	24	40
RS-79	Culvert	469792	672455	8.50	8.48	18	40
RS-80	Culvert	479779	676660	9.70	9.68	24	40
RS-81	Culvert	480163	675342	8.50	8.48	24	40
RS-82	Culvert	480559	674024	8.68	8.66	24	40
RS-83	Culvert	480245	672700	9.00	8.98	24	40
RS-84	Culvert	480317	671383	8.70	8.68	24	40
RS-85	Culvert	480346	670060	8.40	8.38	24	40
RS-86	Culvert	480693	668744	8.50	8.48	24	40

Figure 55. Scenario 3 – Existing Cross Drains

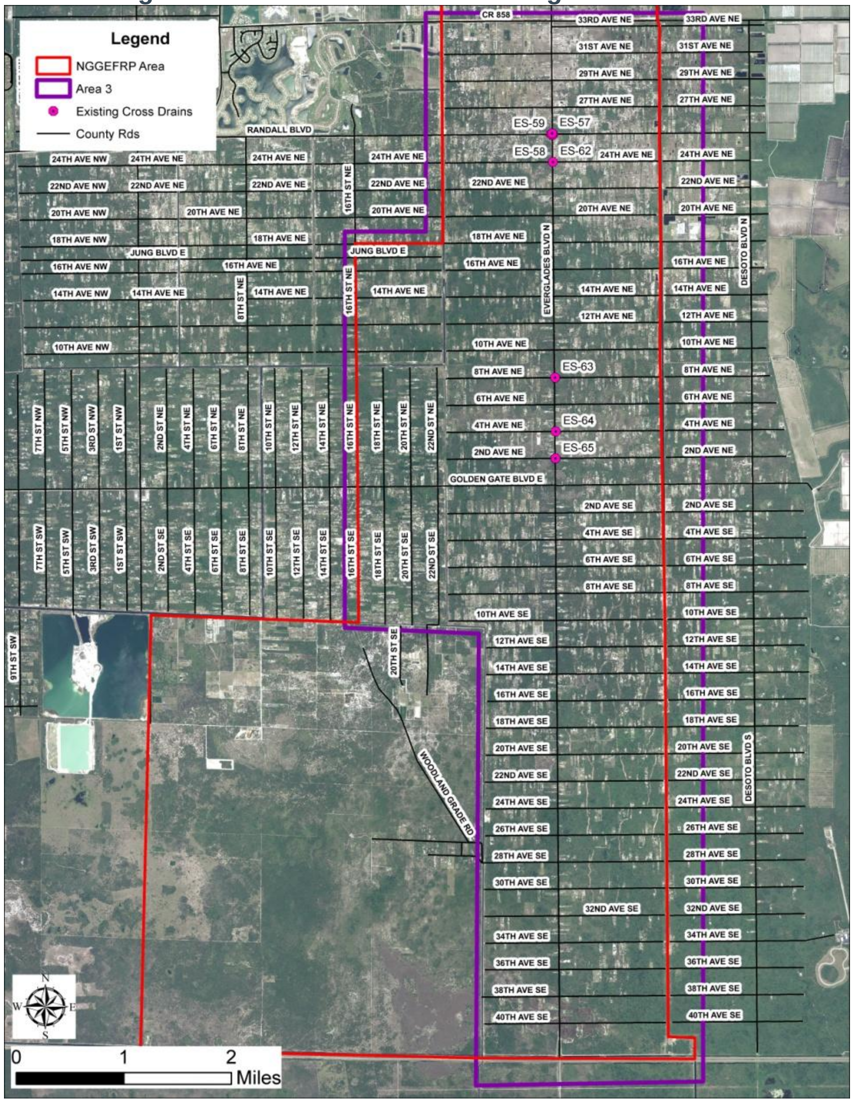
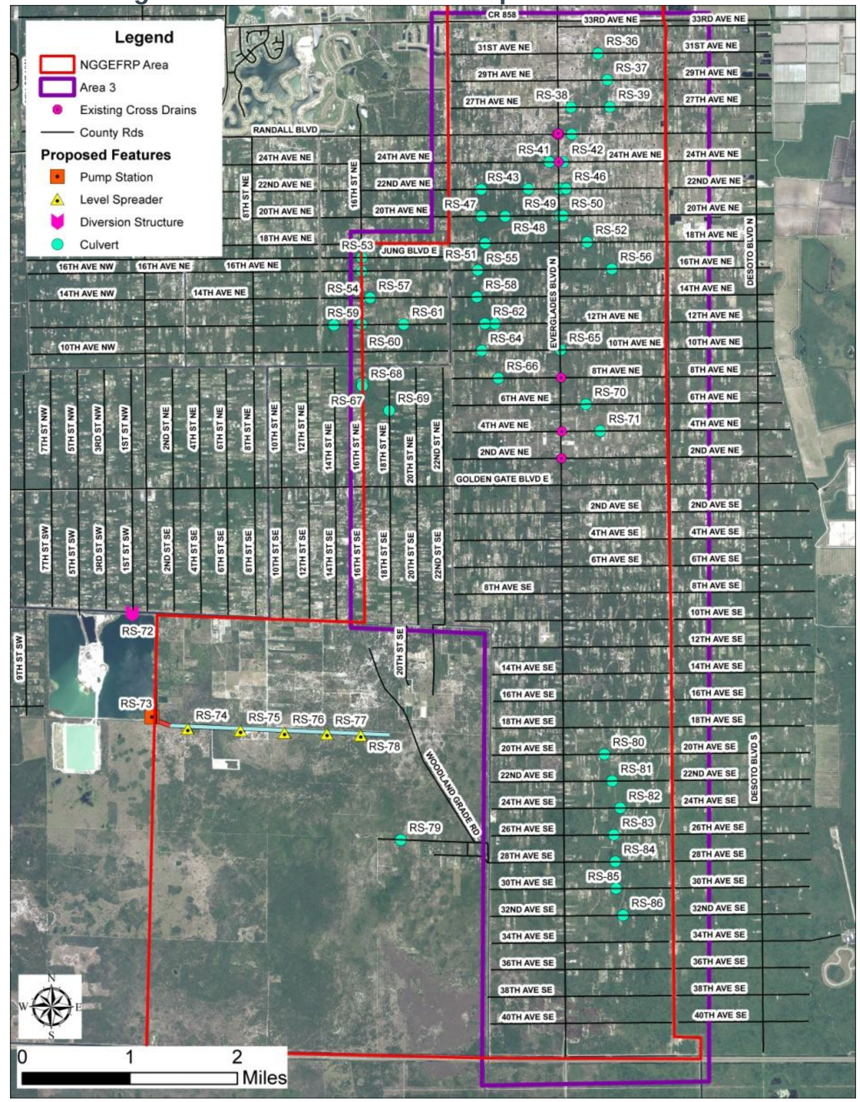


Figure 56. Scenario 3 – Proposed Cross-Drains



3.6.2. Scenario 3 – Hydrologic and Hydraulic Analysis

This section summarizes the results of the simulations completed to evaluate the effects of installing the proposed cross-drains described in **Table 17**. This section discusses the results for each of the following issues:

- Flow in the Miller and Faka Union Canals at I-75.
- Predicted Change in Hydroperiod
- Predicted Change in Seasonal Average and High Water Levels

3.6.3. Flow in the Miller and Faka Union Canals at I-75

Figures 57 and 58 present comparisons of flow in the Miller and Faka Union Canals at I-75. These locations are at the southern end of the Scenario 3 Area. In the Miller Canal, the graph in **Figure 57** suggests that there is little difference in flow resulting from the proposed improvements. On the other hand, **Figure 58** shows a decrease in peak flows in the Faka Union Canal with little change in the timing of flow.

Table 18 shows the calculated difference in average flow volume for each month in the water years from 2003 – 2007. In the Miller Canal, the results indicate that there is a slight increase in the flow volume throughout the year. The increase averages less than one (1) percent of total flow volume during the year. This is likely the result of adding cross-drains to the streets north of 8th Avenue NE. These cross-drains divert water that currently drains to the Golden Gate Canal, south toward the Miller Canal.

In the Faka Union Canal, the results indicate that the proposed cross-drains contribute to an overall reduction in wet season flows, and a slight increase in early dry season flows. These results are consistent with increased infiltration and later baseflow to the canal network.

Table 18. Scenario 3 Flow Volume Comparison

Average Monthly Flow Volume (million gallons)						
Month	Miller Canal at I-75			Faka Union Canal at I-75		
	Modified Existing	Scenario 3	Difference	Modified Existing	Scenario 3	Difference
January	425	431	6	131	133	2
February	163	167	3	104	105	1
March	126	127	1	82	82	0
April	30	30	0	34	34	0
May	21	21	0	27	27	0
June	974	978	5	586	565	-21
July	1598	1612	15	973	944	-29
August	2800	2820	19	1869	1766	-103
September	2638	2667	29	1865	1790	-75
October	1763	1781	19	1041	1034	-7
November	1445	1456	11	440	445	5
December	998	1010	12	252	255	3

Figure 57. Flow Comparison - Miller Canal at I-75

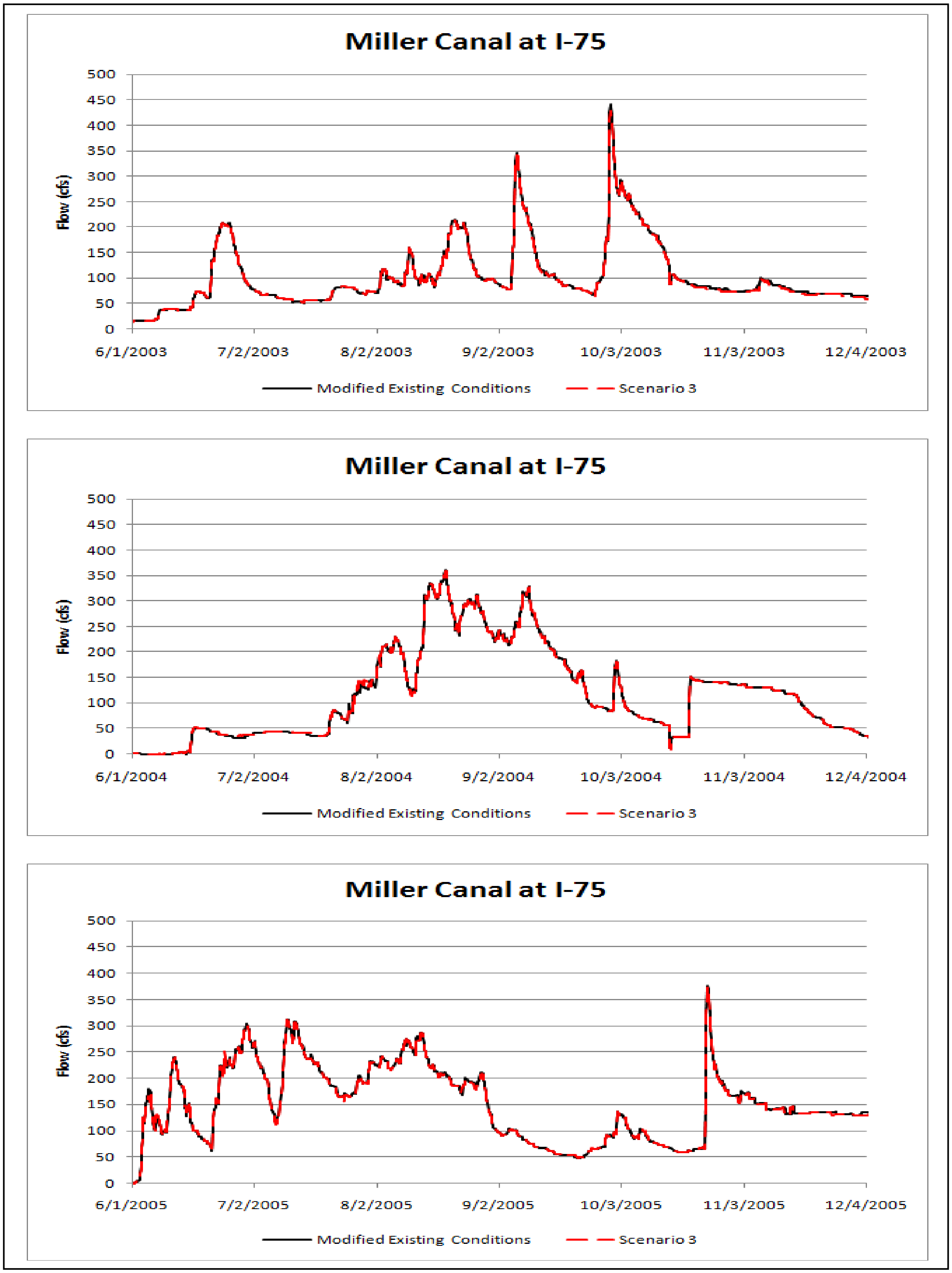
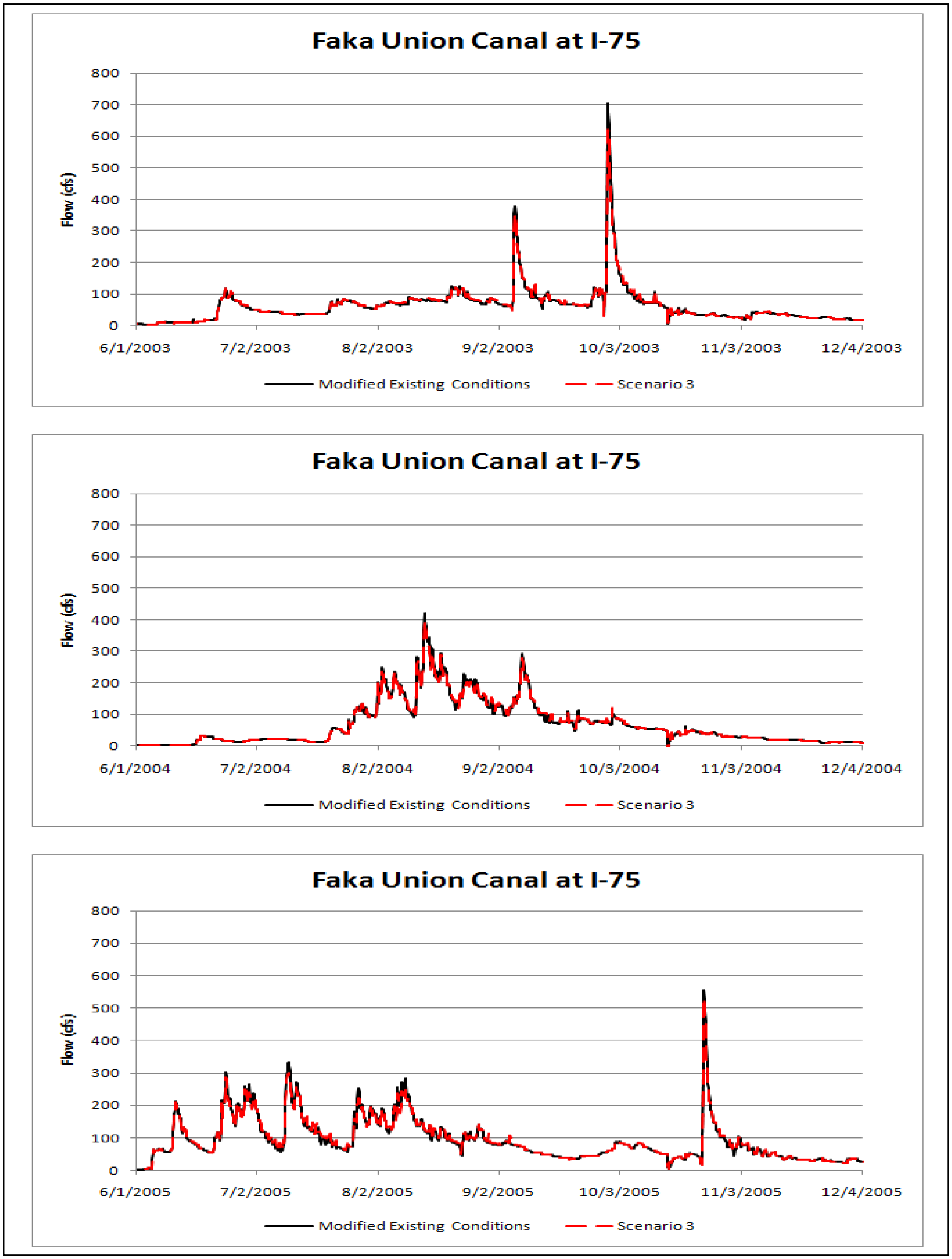


Figure 58. Flow Comparison - Faka Union Canal at I-75



3.6.4. Predicted Change in Hydroperiod

Figure 59 presents a comparison of predicted hydroperiod maps in the Scenario 3 area. The results of the Scenario 3 simulation indicate that there is a decrease in hydroperiod of approximately 45 days in the area immediate south of CR858 (Oil Well Rd), east of Everglades Blvd. The results also show on increase in hydroperiod ranging from 5 to 40 days east of Everglades Blvd in the area between 20th Avenue NE and 2nd Avenue SE.

The results also indicate a 20 – 40 day increase in hydroperiod west of Everglades Blvd along 6th Avenue NE with no apparent change in the area immediately to the north.

There is no obvious increase in hydroperiod in the portion of the Scenario 3 area west of the Golden Gate and Miller Canals along 16th Street NE. This suggests that the cross-drains in this area provide no hydrologic benefit.

In the southern part of the Scenario 3 area, it appears that the hydroperiod had decreased by 10 - 15 days near 22nd Avenue SE and increased by 10 – 20 days further south around 34th Avenue SE.

As in the Scenario 2 area, it appears that the inclusion of the cross-drains has allowed surface water to migrate in a southerly direction. This may have the benefit of increasing recharge within the Collier County wellfield area.

3.6.5. Predicted Change in Seasonal Average and High Water Level

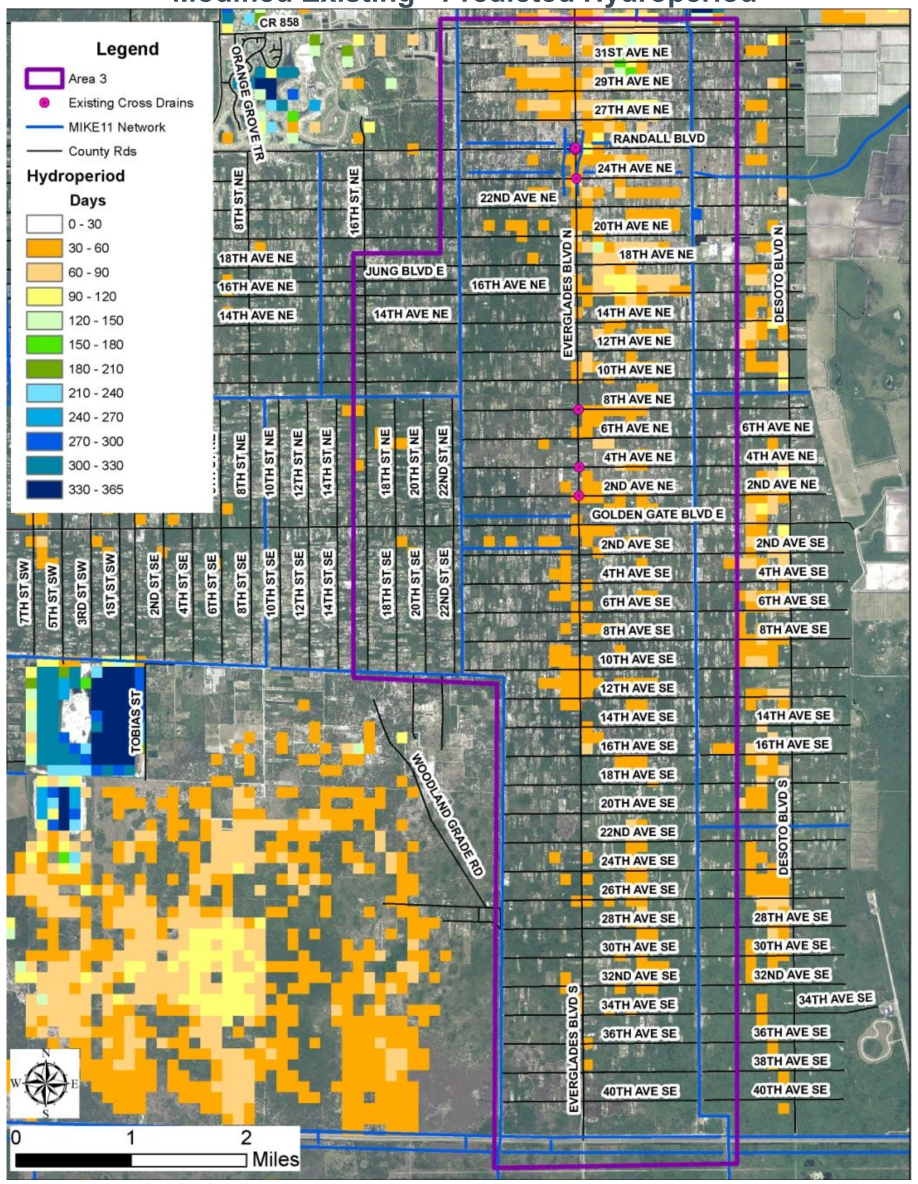
The difference in Average Wet Season Groundwater Elevation and the Seasonal High Groundwater Elevation are presented in **Figures 60 and 61**. In both figures, a positive value indicates that the groundwater elevation is higher in the Scenario 3 simulation; whereas, a negative value indicates that the groundwater elevation is lower.

The results in **Figures 60 and 61** both indicate that the Scenario 3 simulation predicts a rise in wet season groundwater elevation of approximately five (5) inches in the area west of Everglades Blvd, near the Miller and Golden Gate Canals. There is a similar decrease in groundwater elevation immediately south of Oil Well Rd suggesting that water is flowing in a southwestern direction through the proposed cross-drains. Much of the area between 18th Avenue NE and Golden Gate Blvd shows an increase in average groundwater elevation of 1 – 2 inches and that corresponds to an increased hydroperiod of 30 - 60 days.

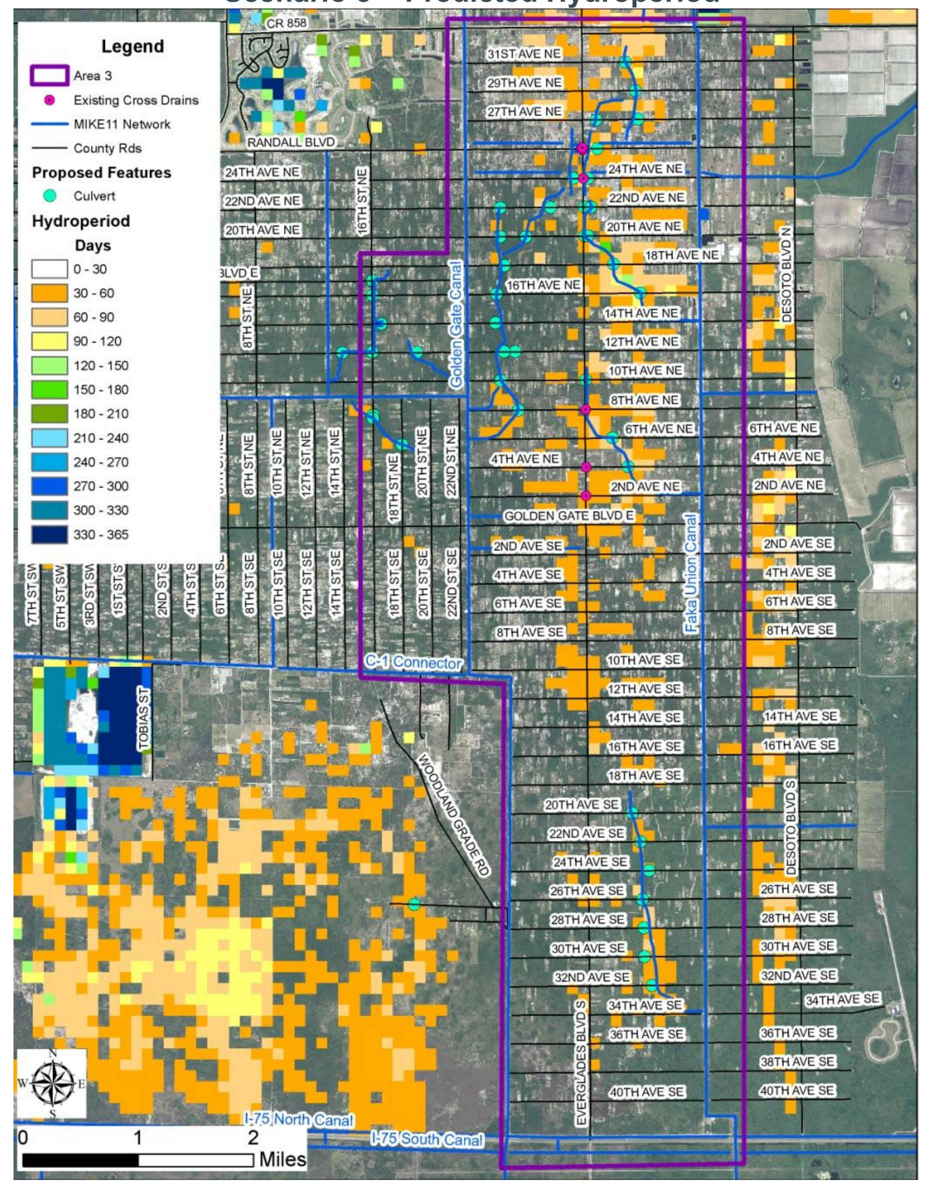
Similarly, there is an increase in the average groundwater elevation of approximately five (5) inches near 34th Avenue SE, east of Everglades Blvd. This corresponds to an area of lower groundwater elevation immediately to the north of this area.

Figure 59. Hydroperiod Comparison:

Modified Existing - Predicted Hydroperiod



Scenario 3 – Predicted Hydroperiod



Scenario 3 – Difference in Predicted Hydroperiod

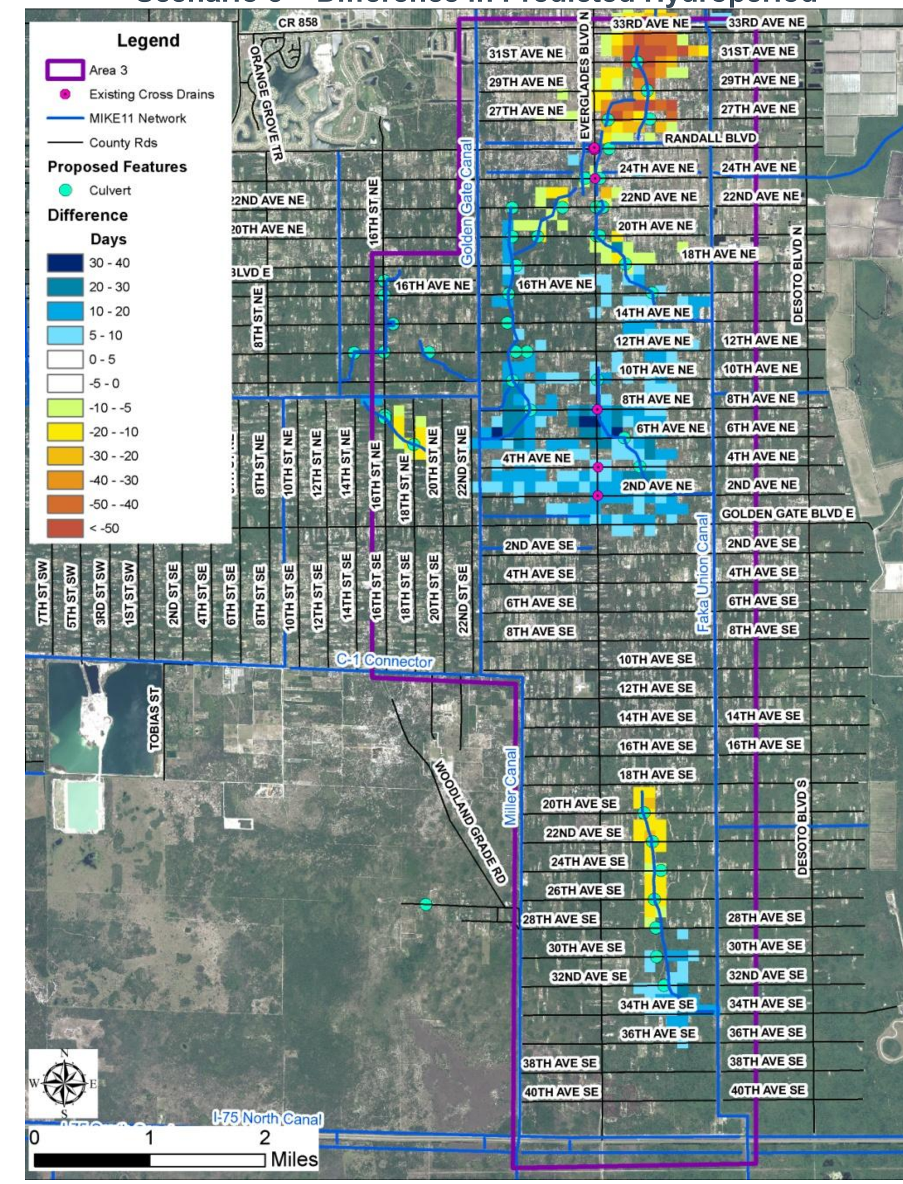


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Figure 60. Scenario 3 – Difference in Average Wet Season Groundwater Elevation

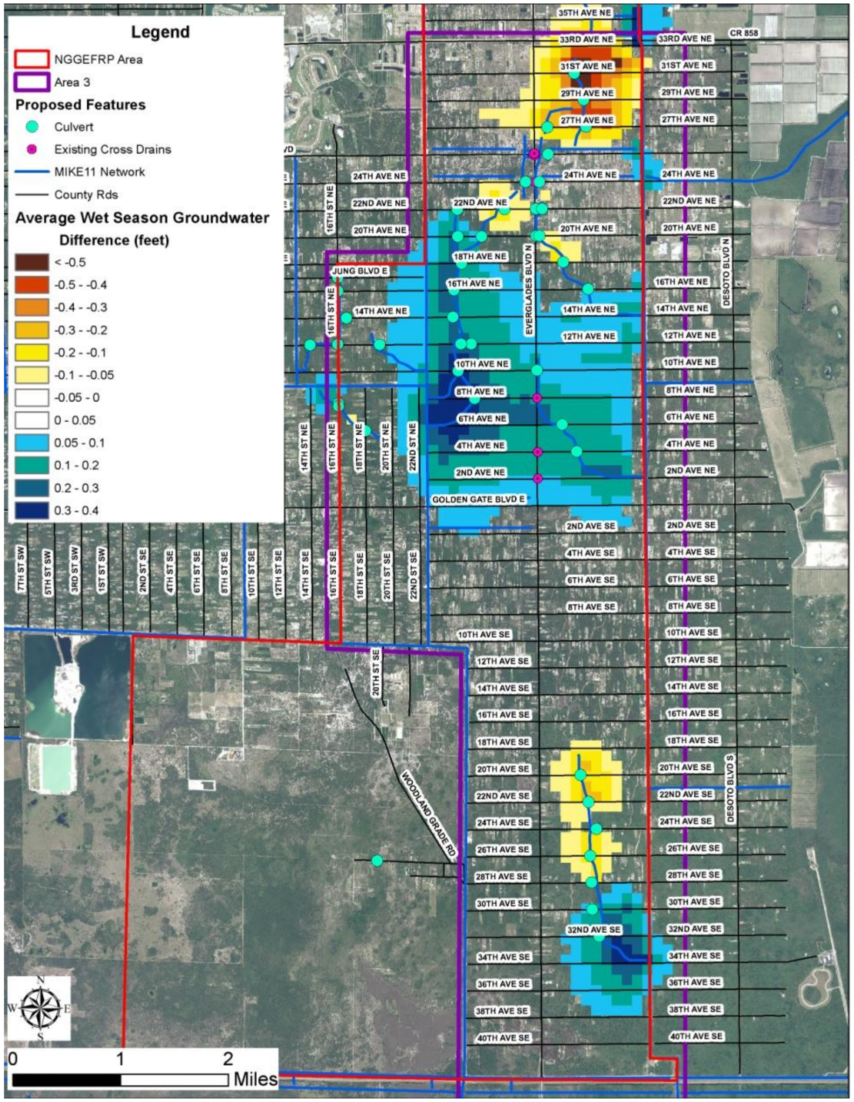
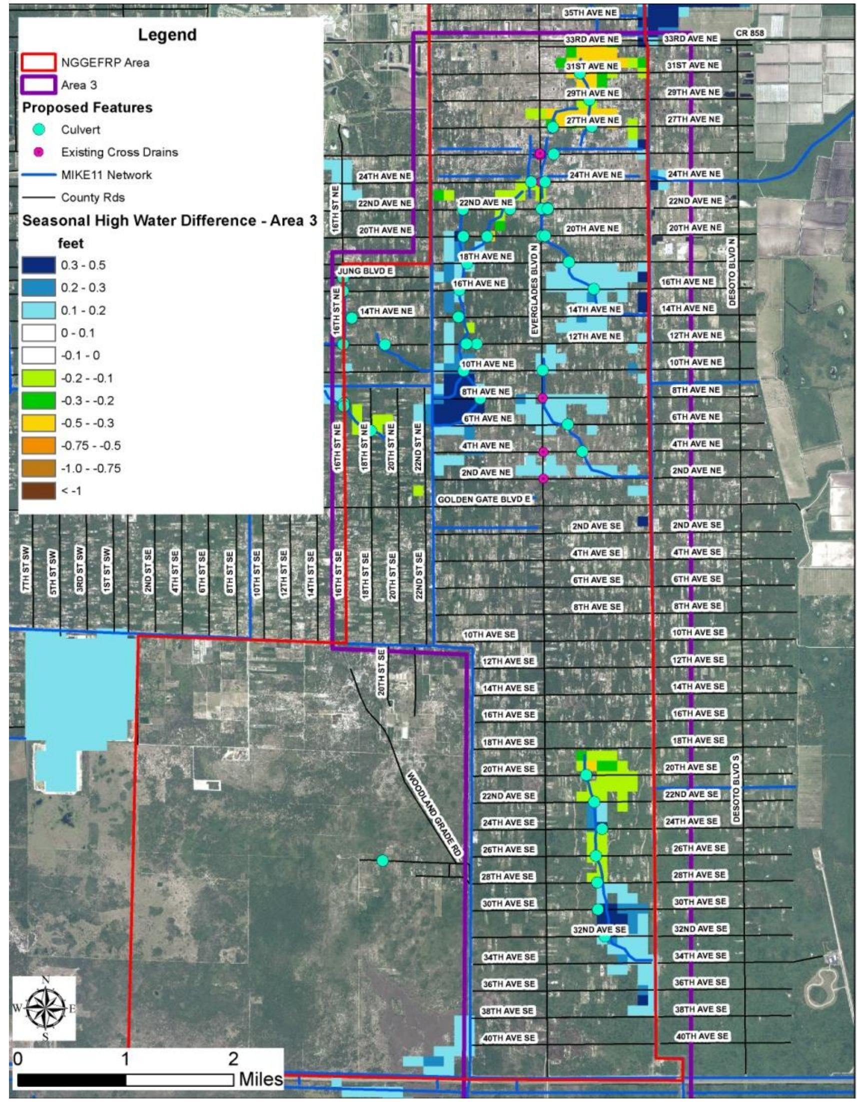


Figure 61. Scenario 3 – Difference in Seasonal High Groundwater Elevation



The areas with increases in groundwater elevation of approximately five (5) inches near 8th Avenue NE and 34th Avenue SE may encroach upon septic leach fields as shown in **Figures 62 - 64**. These results suggest indicate that most of the homes with more elevated construction pads would not be affected by the predicted changes in groundwater elevation. However, there are a few homes where the change in groundwater elevation may impact the septic system leach field. One example is the 4th homepad from the left in **Figure 62**. For that home, it appears that the top of pad elevation meets the minimum septic tank leach field requirements in the modified existing condition. However, the predicted increase in groundwater elevation in the proposed condition may violate the minimum requirements.

Figure 62. Seasonal High Groundwater Elevation South Side of 10th Avenue NE from Golden Gate Canal to Everglades Blvd.

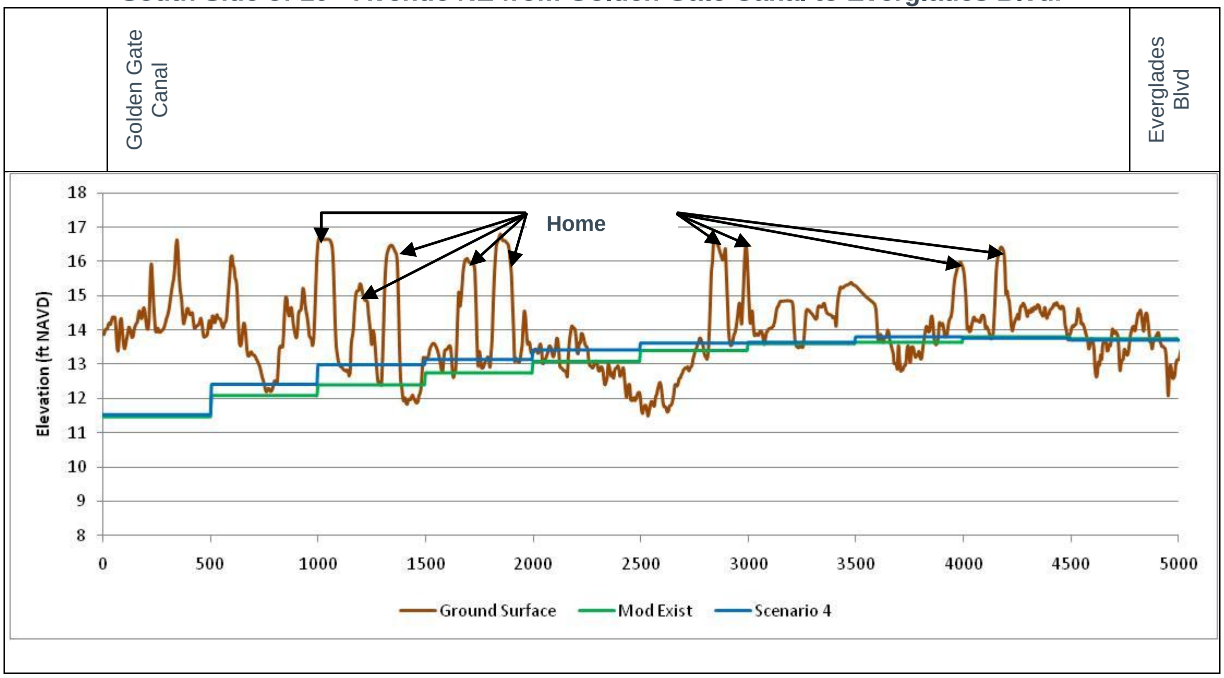


Figure 63. Seasonal High Groundwater Elevation
North Side of 8th Avenue NE from Golden Gate Canal to Everglades Blvd.

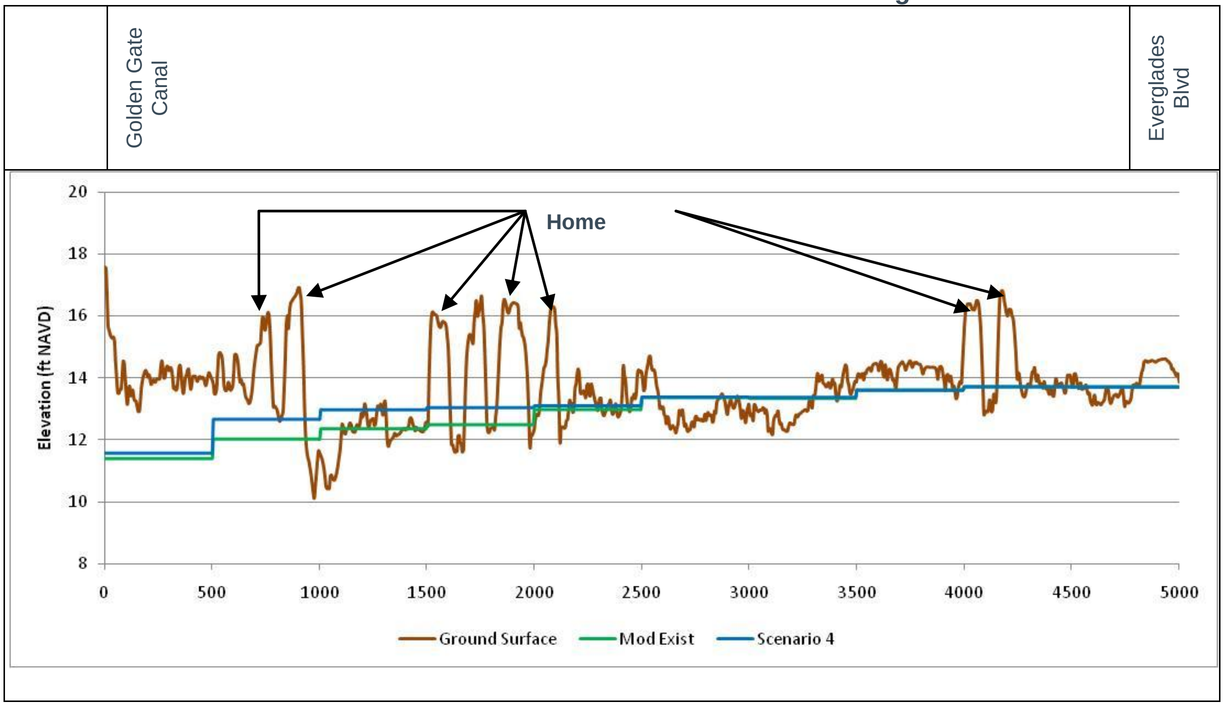
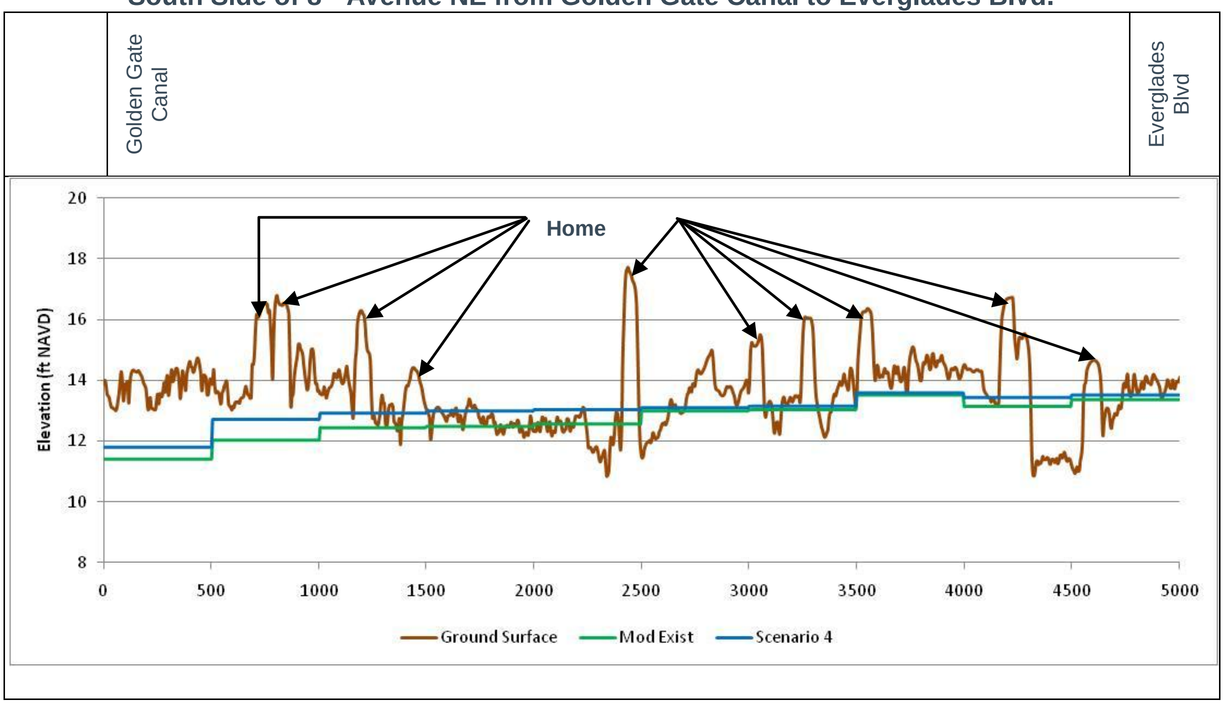


Figure 64. Seasonal High Groundwater Elevation
South Side of 8th Avenue NE from Golden Gate Canal to Everglades Blvd.



3.6.6. Scenario 3 – Conclusions

The following conclusions were drawn from the analysis completed for Scenario 3.

- The addition of the proposed cross-drains in the Scenario 3 contributes to a rise in seasonal average and seasonal high groundwater elevation near the County well fields. This will likely result in increased recharge to the well field.
- The addition of the proposed cross-drains appears to divert water from the Golden Gate Canal toward the Miller Canal near 8th Avenue NE, west of Everglades Blvd. This would contribute to reduced flows to Naples Bay.
- There is no perceived benefit to adding cross-drains in the area west of the Miller and Golden Gate Canals along 16th Street NE.
- Model results suggest that homes constructed on relatively low pads may have their septic leach fields affected by the change in the seasonal high groundwater elevation. This is particularly relevant near 8th Avenue NE, west of Everglades Blvd.

The results of the Scenario 3 analysis suggest that adding cross-drains would provide the overall benefit of expanded wetland areas and increases in groundwater elevations near the county wellfield. However, the County would have to consider the consequences to several homes that are constructed on low pads and may be affected by changes in groundwater elevation. It may be possible to fit many of the culverts with operable flap gates or drop structures to minimize impacts on downstream private property.

3.7. Scenario 4 – Combined Projects

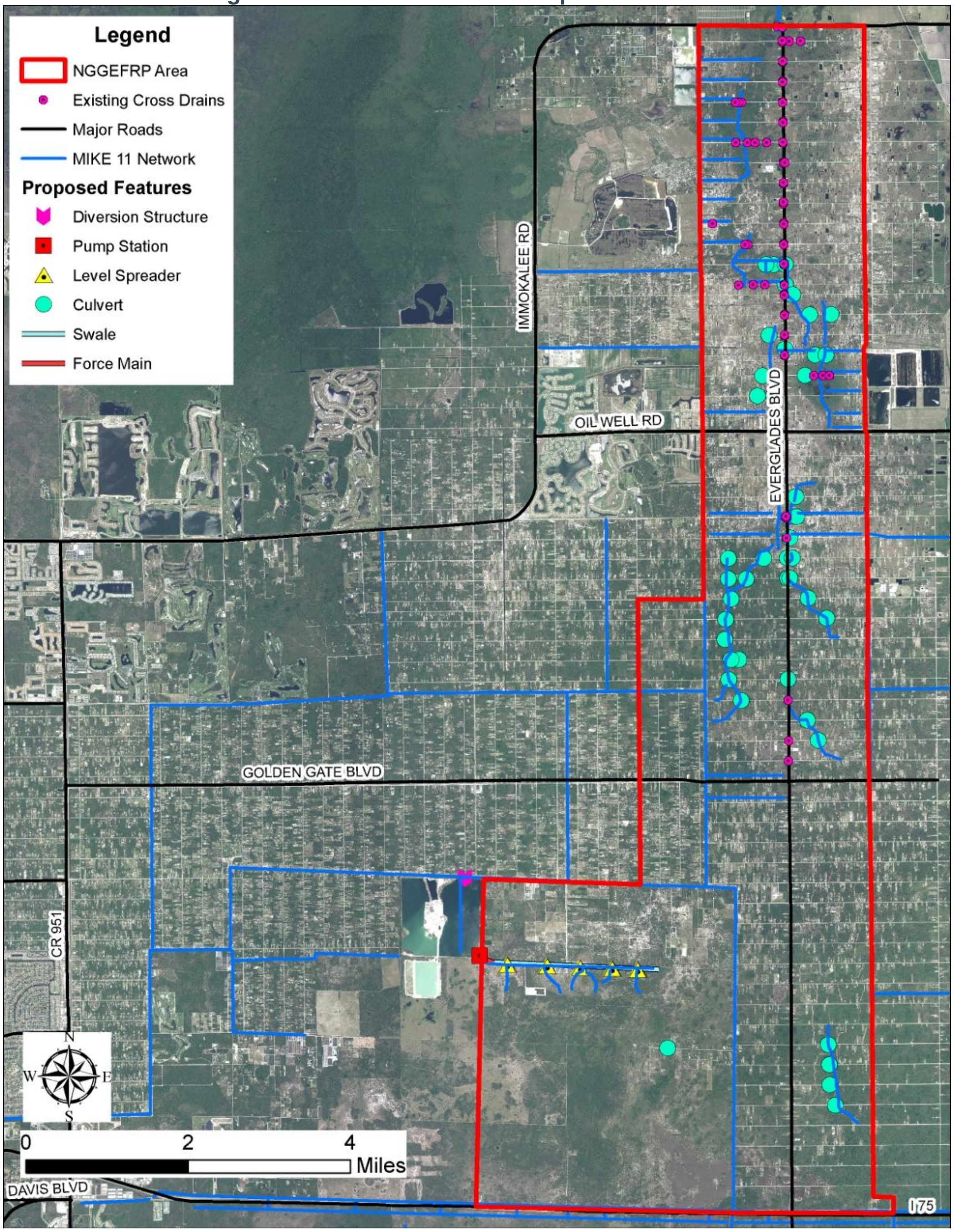
3.7.1. Description of Scenario

Several different model runs were completed for Scenario 4. Each of the simulations considered the removal of proposed cross-drains that appeared to provide negative or no hydrologic benefit. The final Combined Scenario considers the interaction of the system with the following improvements:

- Scenario 1, Option 1 (800 cfs pump) of the North Belle Meade Spreader Swale
- Many of the proposed cross-drains proposed in Scenario 2 were removed from the model. These include all proposed cross-drains in the Panther Walk area and several in the Winchester Head Area.
- Several of the proposed cross-connects described in Scenario 3 were removed from this scenario. These include those immediately south of Oil Well Rd and several in the southern most part of the study area.

Figure 65 shows the final configuration of improvements recommended for the Northern Golden Gates Estates Area.

Figure 65. Scenario 4 – Proposed Features.



3.7.2. Scenario 4 – Hydrologic & Hydraulic Analysis

This section summarizes the results of the simulations completed to evaluate the effects of installing all of the projects identified in Scenario 4. This section discusses the results for each of the following issues:

- Flow and Stage in the Golden Gate Canal.
- Flow and Stage in the Miller Canal
- Flow and Stage in the Faka Union Canal
- Predicted Change in Depth of Flood Inundation
- Predicted Change in Hydroperiod
- Predicted Change in Seasonal Average and High Water Levels

3.7.3. Flow and Stage in the Golden Gate Canal

Table 19 presents a comparison of predicted flows over the GG-3 structure resulting from Scenario 4. The results are very similar to the Scenario 1, Option 1 results throughout the simulation period. However, the Scenario 4 results generally predict slightly less flow throughout the year. Compared to the Scenario 1, Option 1 results, the greatest additional reductions in flow occur in July, November, and December.

The additional reduction in flow in July can be attributed to the cross-drains in the Northern Golden Gate Estates area. The cross-drains and diversion structure do not effect water levels in the Golden Gate Canal during the dry season (**Figure 66**), but do contribute to an overall reduced volume of runoff discharging from the Golden Gate Canal and provide an opportunity for increased infiltration. Reduced flows in November and December are attributable to additional storage created by lower water levels in the Golden Gate Canal between the GG-3 and GG-4 structures (**Figure 67**) during the wet season. The Scenario 4 result predicts that the water level between these structures will be approximately six (6) inches lower at times during the wet season. This means that there is more storage available upstream of the GG-3 structure and subsequently contributes to lower total discharge volumes later in the year.

Table 19. Flow Comparison - Golden Gate Canal at GG-3 Structure

	Average Monthly Flow Volume (million gallons)			
	GG-3 Structure			
Month	Modified Existing	Scenario 1 Option 1	Scenario 4 800 cfs pump	Difference S1 – S4
January	315	282	281	1
February	325	319	324	-5
March	371	367	368	-1
April	32	31	31	0
May	56	57	57	0
June	2390	2122	2128	-6
July	4226	3842	3794	48
August	6887	5673	5704	-31
September	7161	6102	6059	43
October	5140	4750	4749	1
November	2116	2028	1999	29
December	711	667	655	12

Figure 66. Surface Water Profile of Golden Gate Canal in the Middle of the Dry Season (2-1-2004)

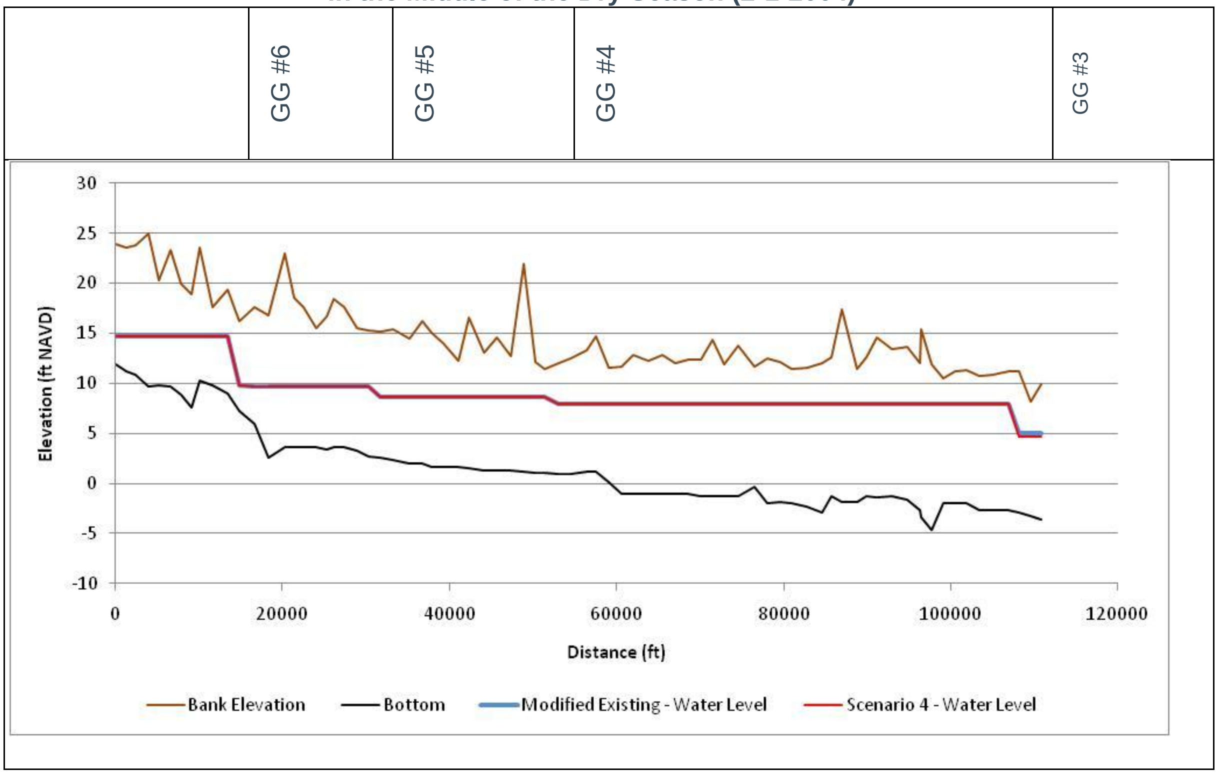
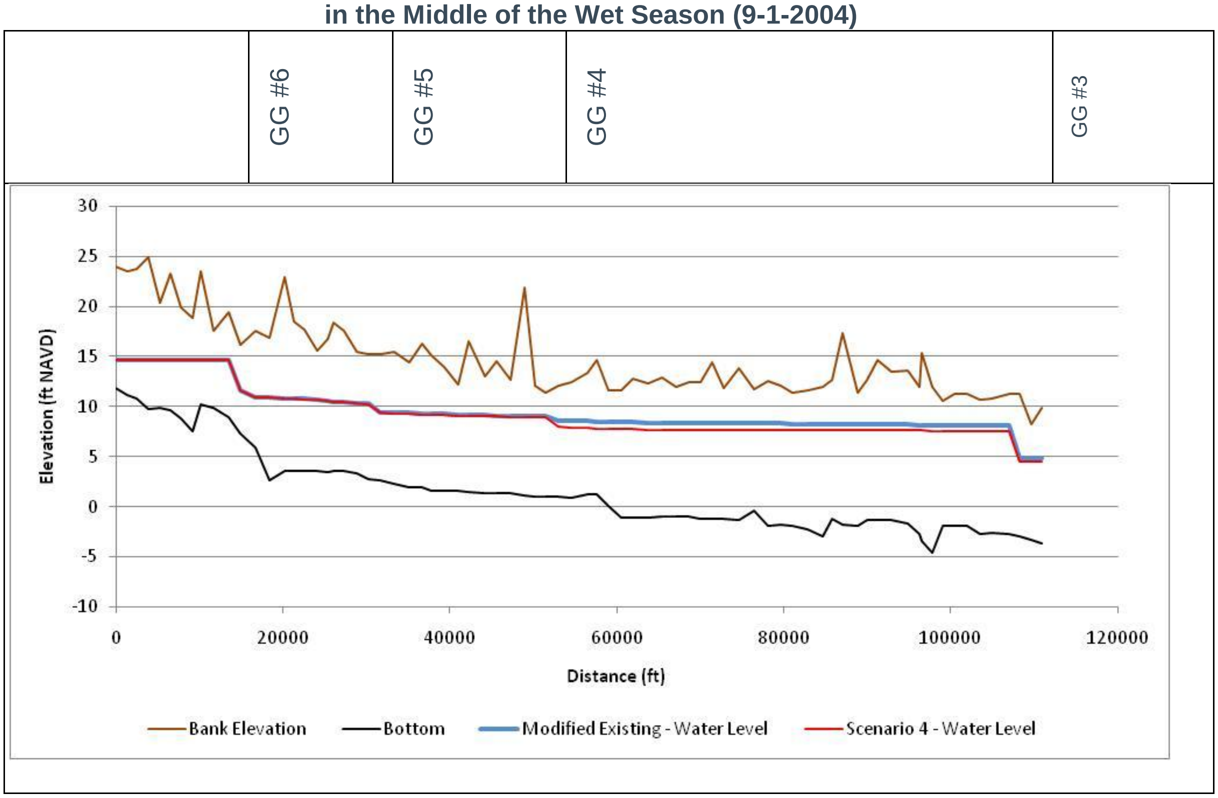


Figure 67. Surface Water Profile of Golden Gate Canal in the Middle of the Wet Season (9-1-2004)



3.7.4. Flow and Stage in the Miller Canal

The information in **Table 20** was extracted from a location in the model results immediately south of I-75. These results indicate that the combined volume of water discharged by the PSRP - Miller Canal Pump Station is slightly higher than the Scenario 1, Option 1 results during the wet season and the early dry season. The late dry season results are almost identical to the Modified Existing simulation results. The increased wet season pumping can be attributed to the inflow contributions resulting from the Scenario 3 cross-drains allowing water that would typically drain to the Golden Gate Canal to flow toward the Miller Canal. Additional early dry season pumping is linked to the higher water levels in the Miller Canal that are drawn down over a longer period of time.

Profile views of water level in the Miller Canal from the intersection with the Golden Gate Canal to the Miller Canal Pump Station are found in **Figures 68 and 69**. The water levels are approximately equal during the middle of the dry season when there is little difference in pumped volume. However, the wet season water level is almost one (1) foot higher at a location just north of I-75 due to inflows from the North Belle Meade area via the North I-75 Canal.

Table 20. Flow Comparison - Miller Canal at I-75

	Average Monthly Flow Volume (million gallons)			
Month	Miller Canal at I-75			
	Modified Existing	Scenario 1 Option 1	Scenario 4 800 cfs pump	Difference S1 – S4
January	443	445	458	-13
February	177	171	173	-2
March	137	128	129	-1
April	33	22	22	0
May	20	7	8	-1
June	1187	1307	1298	9
July	1866	2264	2287	-23
August	3667	4690	4715	-25
September	3421	4355	4373	-18
October	2161	2381	2404	-23
November	1605	1614	1639	-25
December	1050	1037	1065	-28

Figure 68. Surface Water Profile of Miller Canal in the Middle of the Dry Season (2-1-2004)

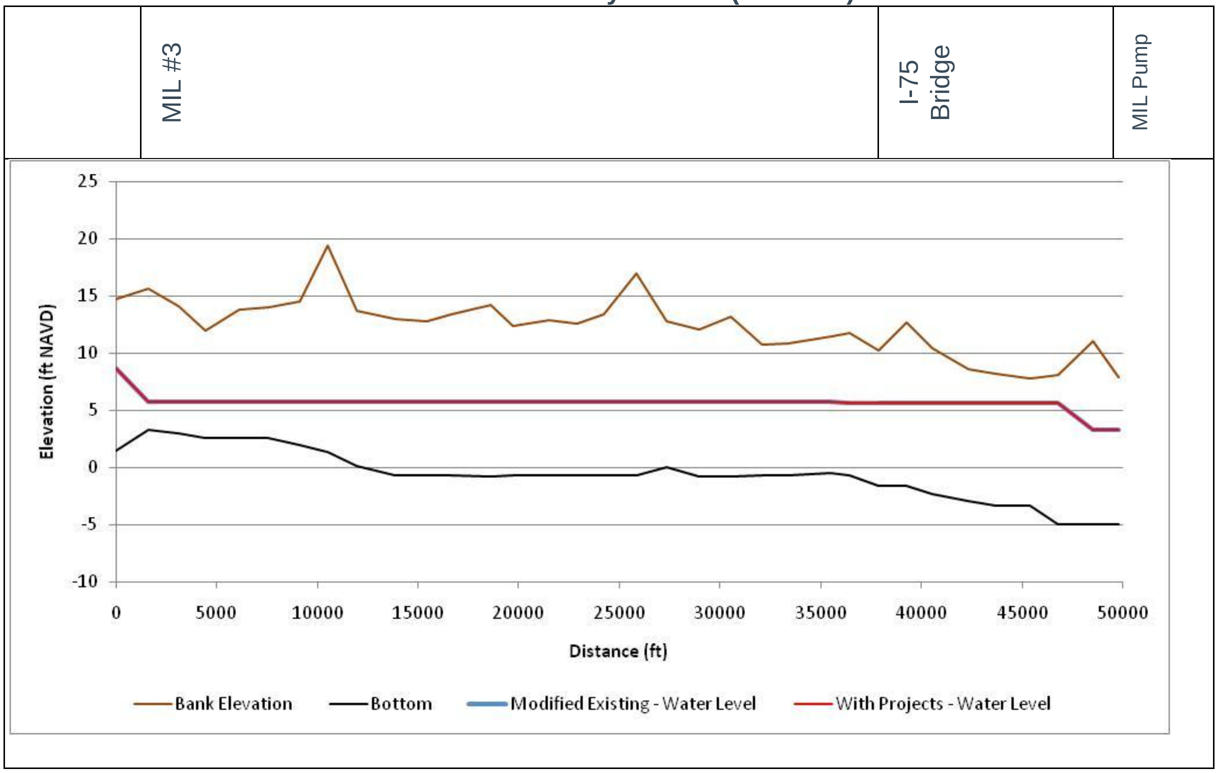
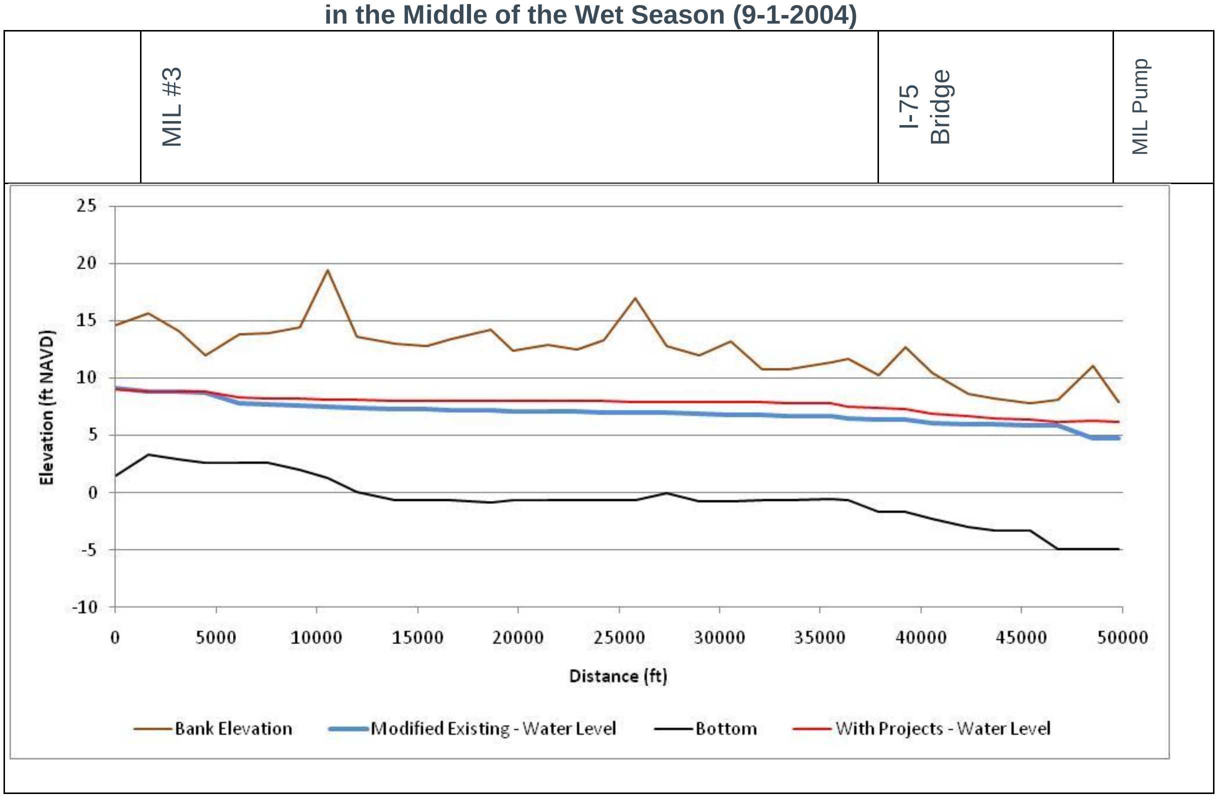


Figure 69. Surface Water Profile of Miller Canal in the Middle of the Wet Season (9-1-2004)



3.7.5. Flow and Stage in the Faka Union Canal

There is little change in flow and stage in the Faka Union Canal resulting from implementation of the proposed improvements. **Table 21** shows that the reduction in flows associated with the combined projects is very similar to that predicted for the Scenario 3 simulation. There continues to be a decrease in wet season flow and a slight increase in early dry season flow consistent with increased recharge to groundwater. **Figures 70 and 71** provide a profile view of the water surface during the wet and dry seasons from the headwater to I-75. In both cases, the water surface elevations are approximately equal for the length of the channel.

Table 21. Flow Comparison - Faka Union Canal at I-75

	Average Monthly Flow Volume (million gallons)			
Month	Faka Union Canal at I-75			
	Modified Existing	Scenario 3	Scenario 4 800 cfs pump	Difference S3 – S4
January	131	133	134	-1
February	104	105	106	-1
March	82	82	83	-1
April	34	34	34	0
May	27	27	27	0
June	586	565	564	1
July	973	944	945	1
August	1869	1766	1786	-20
September	1865	1790	1801	-11
October	1041	1034	1042	-8
November	440	445	447	-2
December	252	255	258	-3

Figure 70. Surface Water Profile of Faka Union Canal in the Middle of the Dry Season (2-1-2004)

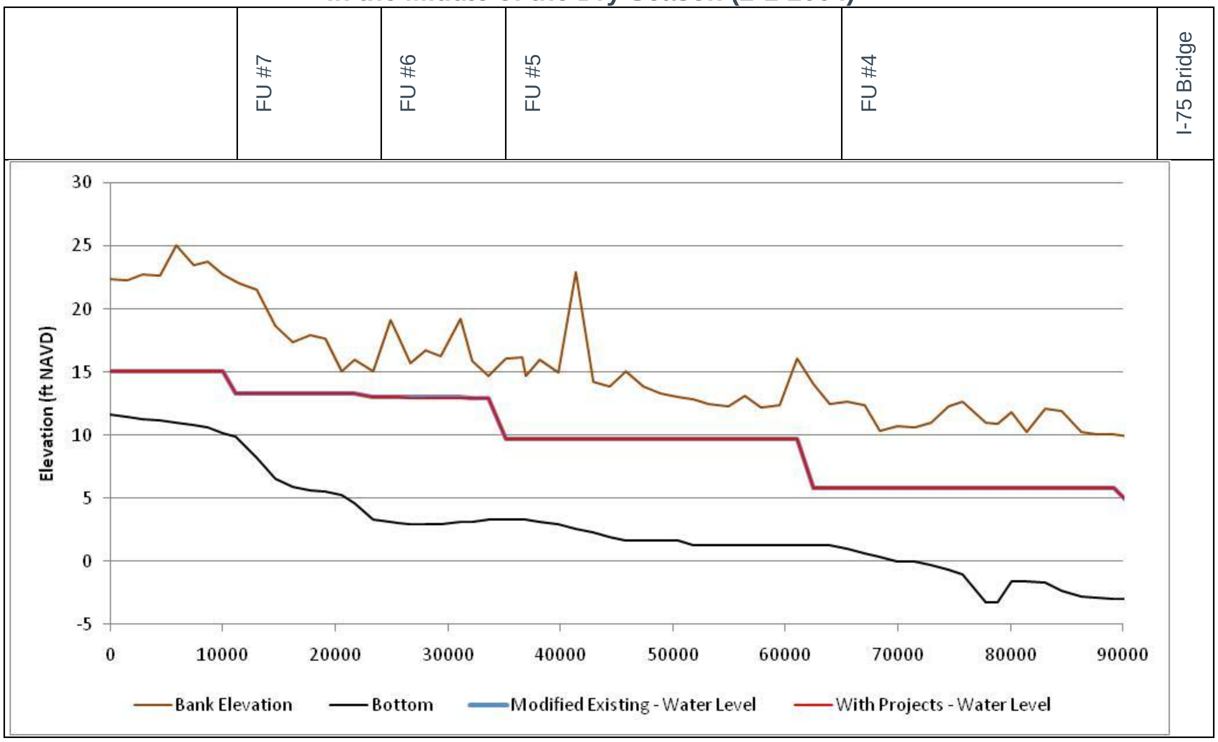
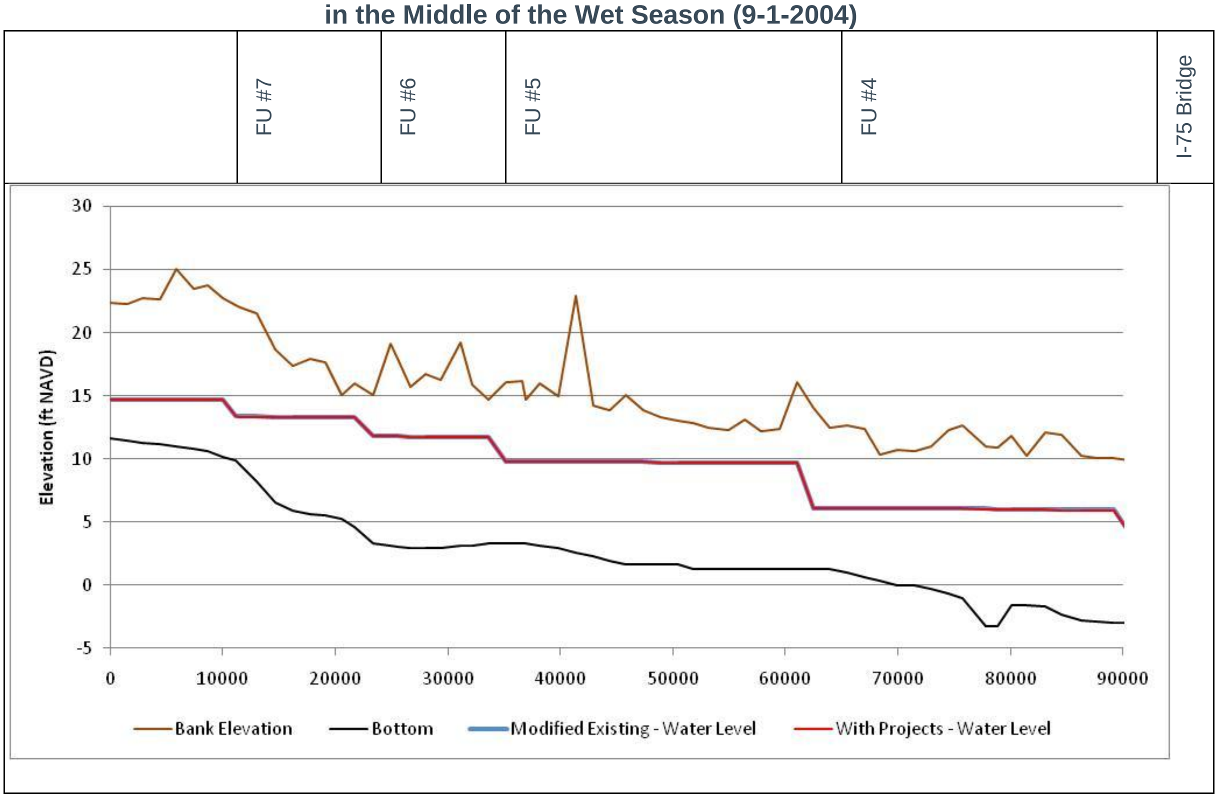


Figure 71. Surface Water Profile of Faka Union Canal in the Middle of the Wet Season (9-1-2004)



3.7.6. Predicted Change in Depth of Flood Inundation

Design storms simulations were completed for the Modified Existing Conditions model and the Scenario 4 model. September 4, 2004 was selected as the hotstart date for these simulations since it represents the date of maximum inundation during the simulation period. Design storm simulations were completed for the 10-year, 25-year, and 100-yr/72 hour storm events. Provide views of the maximum water level in the Golden Gate, Miller, and Faka Union Canals for each design storm are shown in **Figures 72 – 80**.

In the Golden Gate Canal, the predicted water levels in Scenario 4 are lower than the Modified Existing Condition for each of the storm events between the GG3 and GG4 structures. Therefore, there is additional capacity in this portion of the canal during large storm events.

In the Miller Canal, the Scenario 4 water levels are slightly higher along the length of the canal north of I-75. This can be attributed to inflows from the North Belle Meade project and may affect the peak depth of inundation along the canal.

There is a difference in peak water levels in the Faka Union Canal south of the FU4 structure during the 10-year/72 hour storm event. The Scenario 4 water level is lower than in the Modified Existing Conditions indicating that water is held in overland storage. There is no noticeable difference during the 25- and 100-year storm events.

Figures 81 - 83 present a comparison of the maximum depth of inundation across the study area for each of the storm results for the 10-yr, 25-75, and 100-yr/72 hour storm events. Each of these figures represents the Scenario 4 storm result less the Modified Existing Condition storm result. A positive value means that the maximum depth of inundation has increased in Scenario 4, whereas a negative value indicates that the maximum depth of inundation has decreased.

These figures indicate that depth of inundation in the North Belle Meade area south of the spreader swale has increased by as much as 8 inches or more in some locations. There is also in increased depth of inundation (up to 3.5 inches) on the north side of I-75. North of the spreader swale, the depth of inundation is predicted to increase by 1-2 inches, suggesting that the spreader swale may inhibit southerly overland flow.

Each of the figures indicates a decrease in the depth of inundation along the Golden Gate Canal network between the GG-3 and GG-4 structures. Some areas that drain to the Golden Gate Canal are predicted to have a decrease in the depth of inundation of as much as 6 – 8 inches. However, the typical predicted decrease is less than four (4) inches.

The peak depth of flood inundation for Scenario 4 is slightly higher (approximately 1 -2 inches) along the southern portions of the Miller Canal. However, it appears that difference in elevation may influence the depth of inundation east of the canal toward Everglades Blvd. These areas see increases in the depth of inundation ranging from 0.5 – 2 inches. To offset the increased water levels in the Miller Canal, it may be necessary to change the proposed operational rules for the PSRP Miller Canal Pump Station.

Finally, many of the areas in Scenario 4 with proposed cross-drains are predicted to have a higher peak depth of inundation, with most areas predicted to increase by 1 -2 inches. Other areas see a depth of inundation increase of eight (8) or more inches. These areas correspond with the greatest predicted increases in the seasonal groundwater elevation.

Figure 72. Peak Surface Water Profile of Golden Gate Canal 10-year/72-hour Storm Event

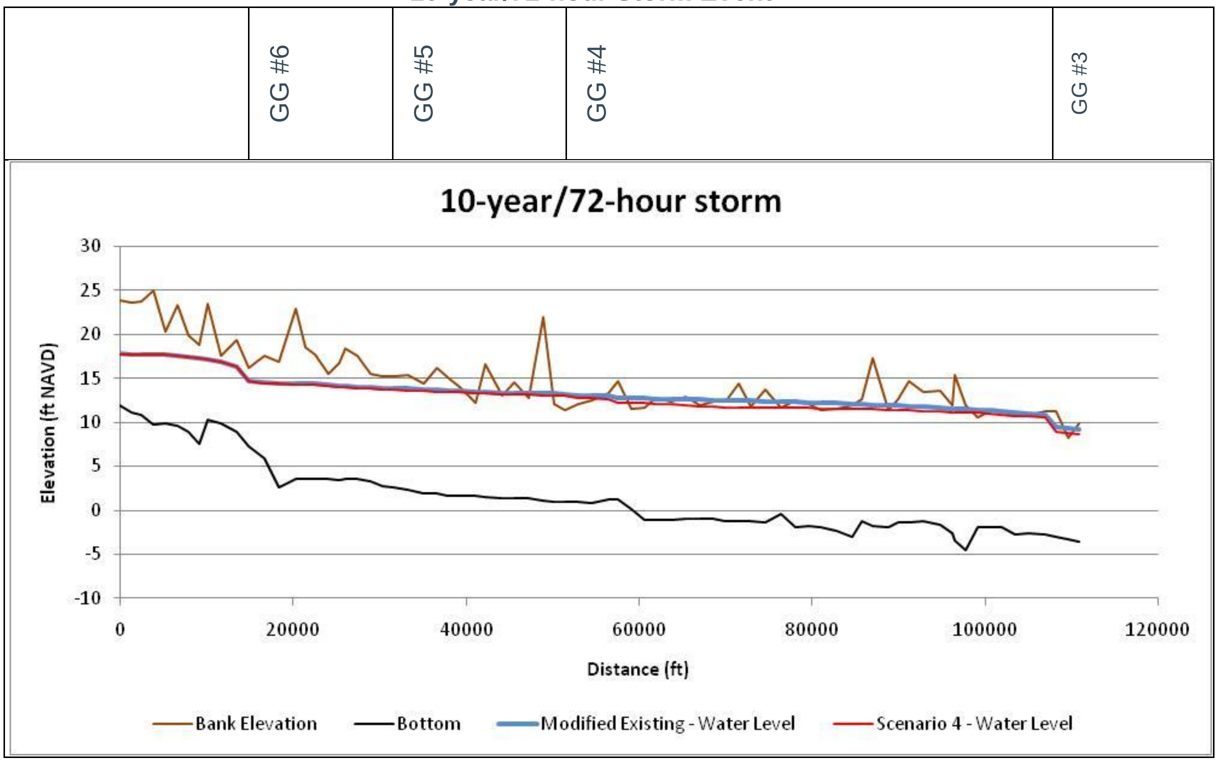


Figure 73. Peak Surface Water Profile of Golden Gate Canal 25-year/72-hour Storm Event

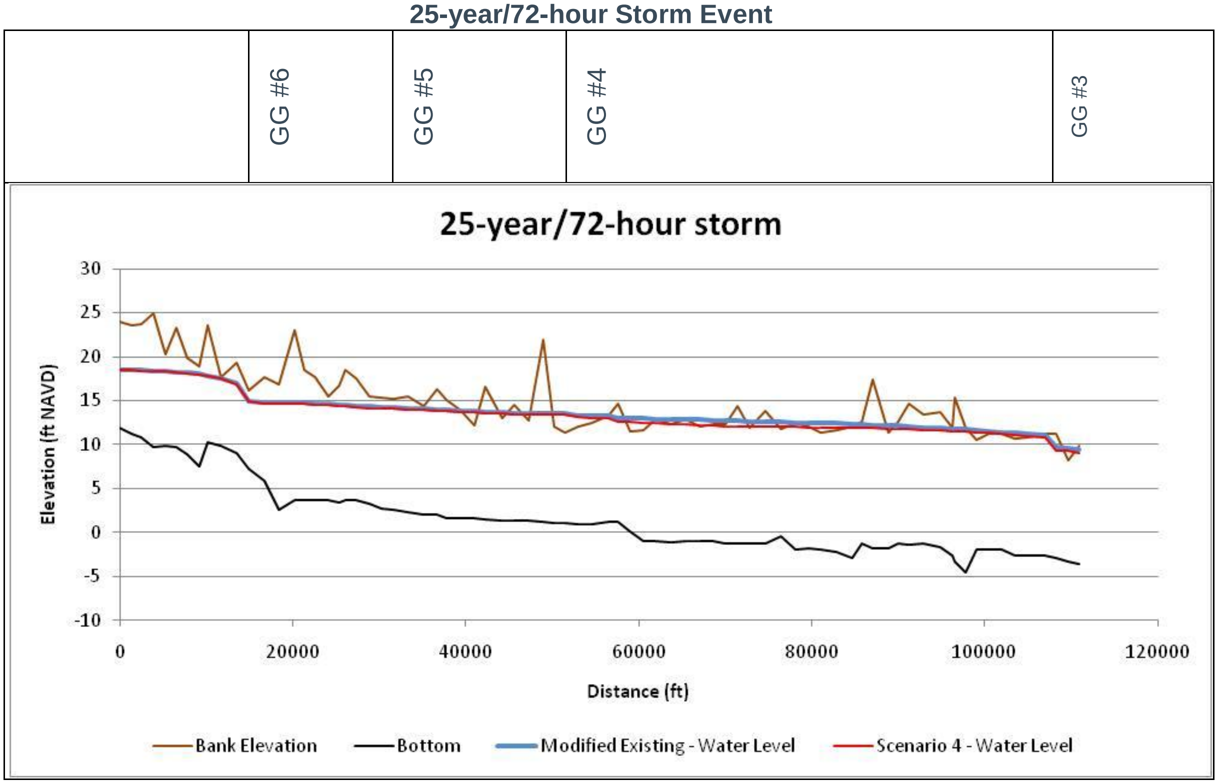


Figure 74. Peak Surface Water Profile of Golden Gate Canal 100-year/72-hour Storm Event

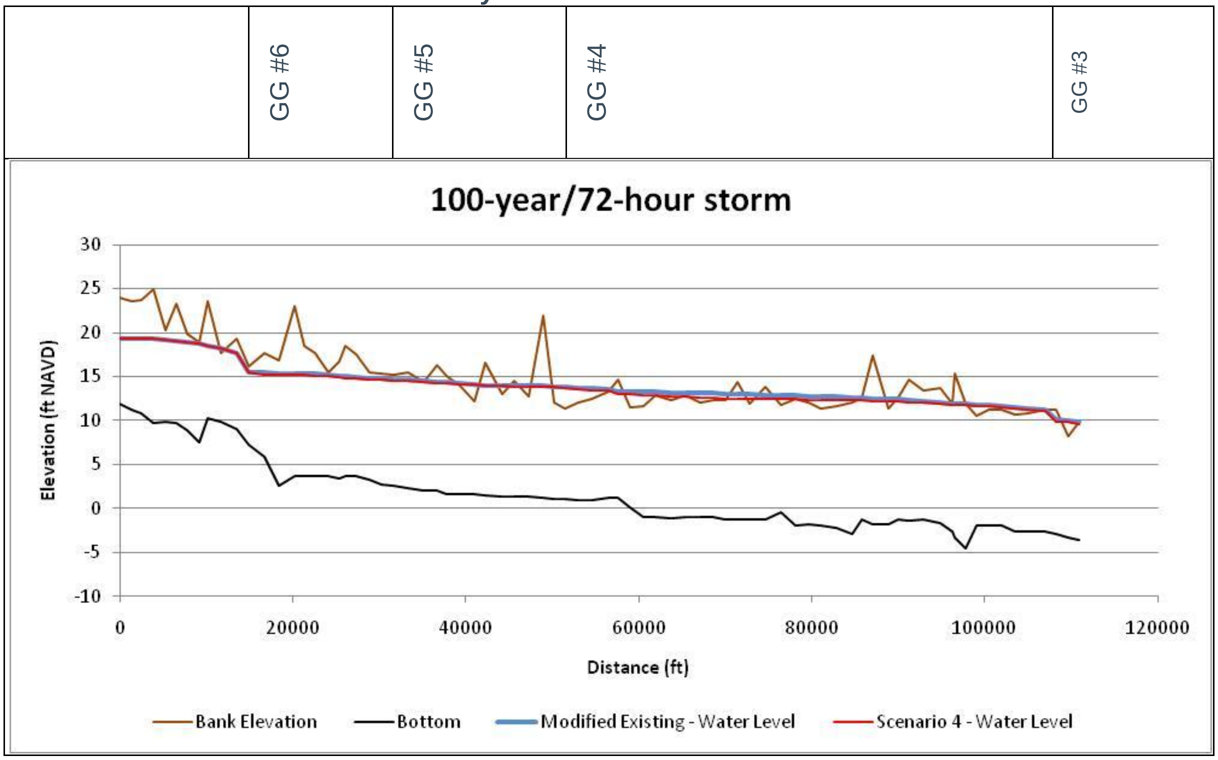


Figure 75. Surface Water Profile of Miller Canal 10-year/72-hour Storm Event

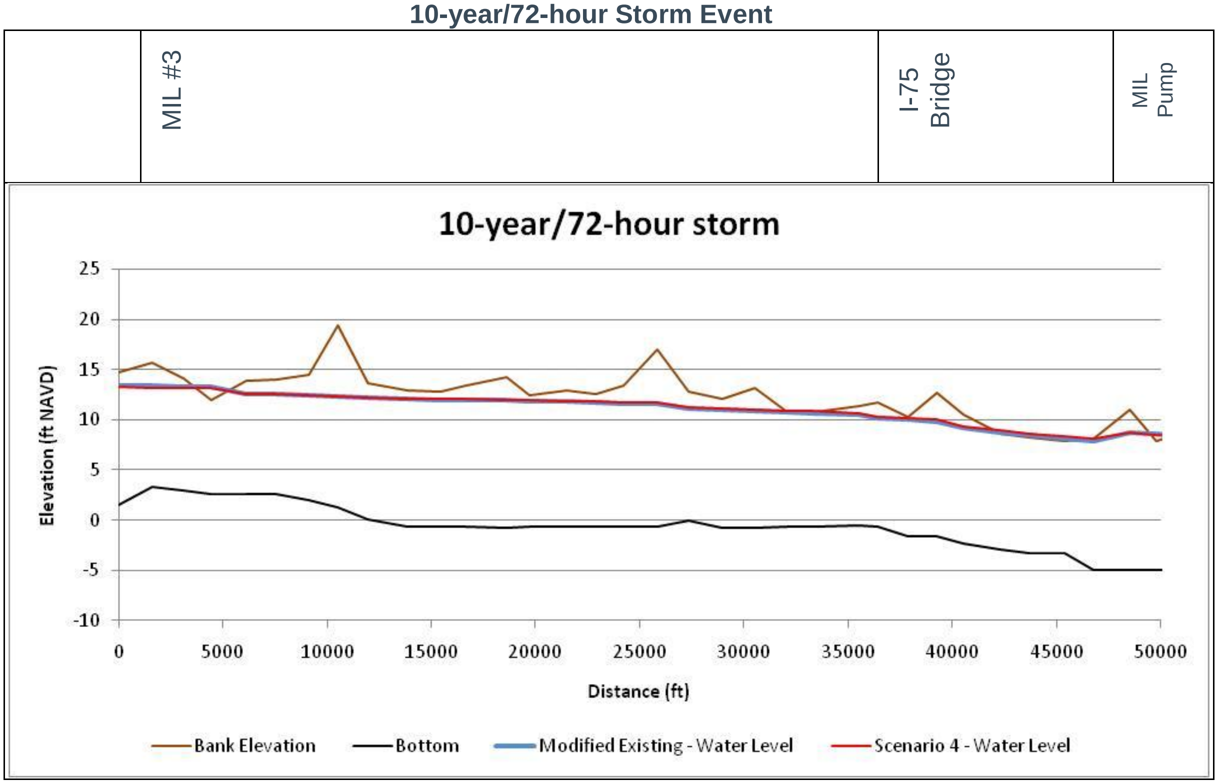


Figure 76. Surface Water Profile of Miller Canal 25-year/72-hour Storm Event

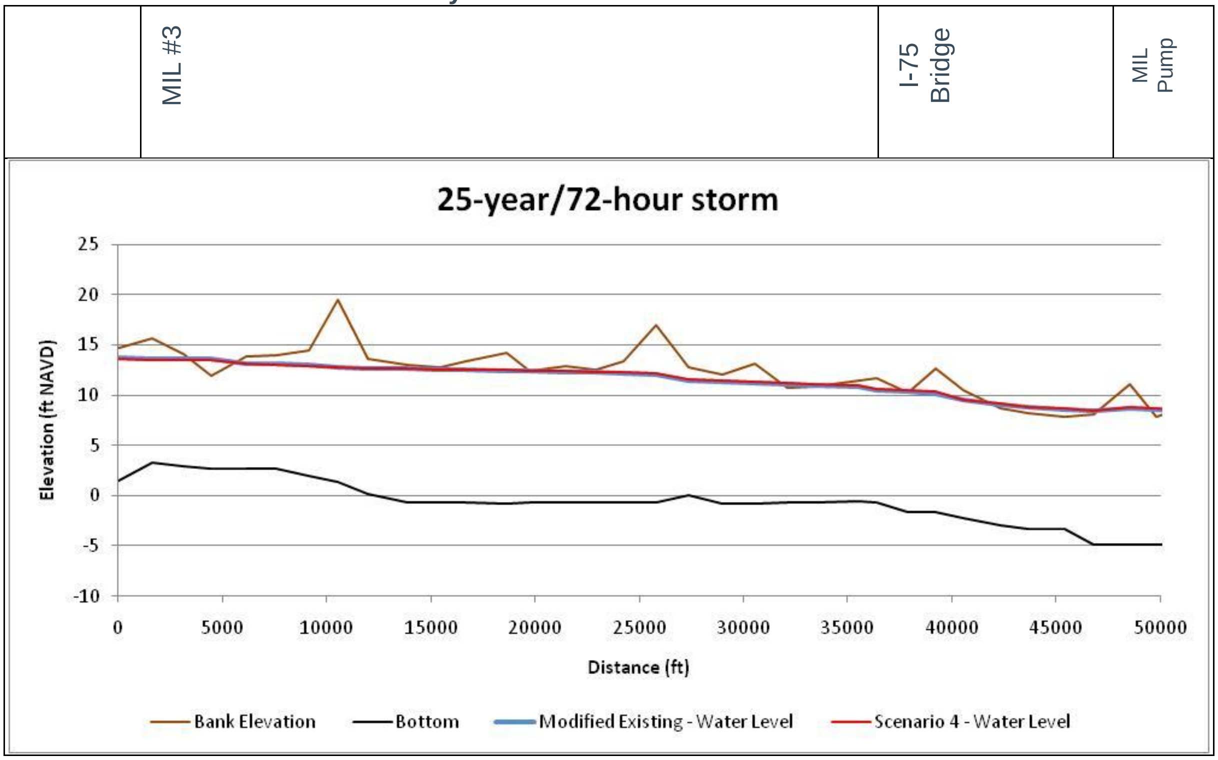


Figure 77. Surface Water Profile of Miller Canal 100-year/72-hour Storm Event

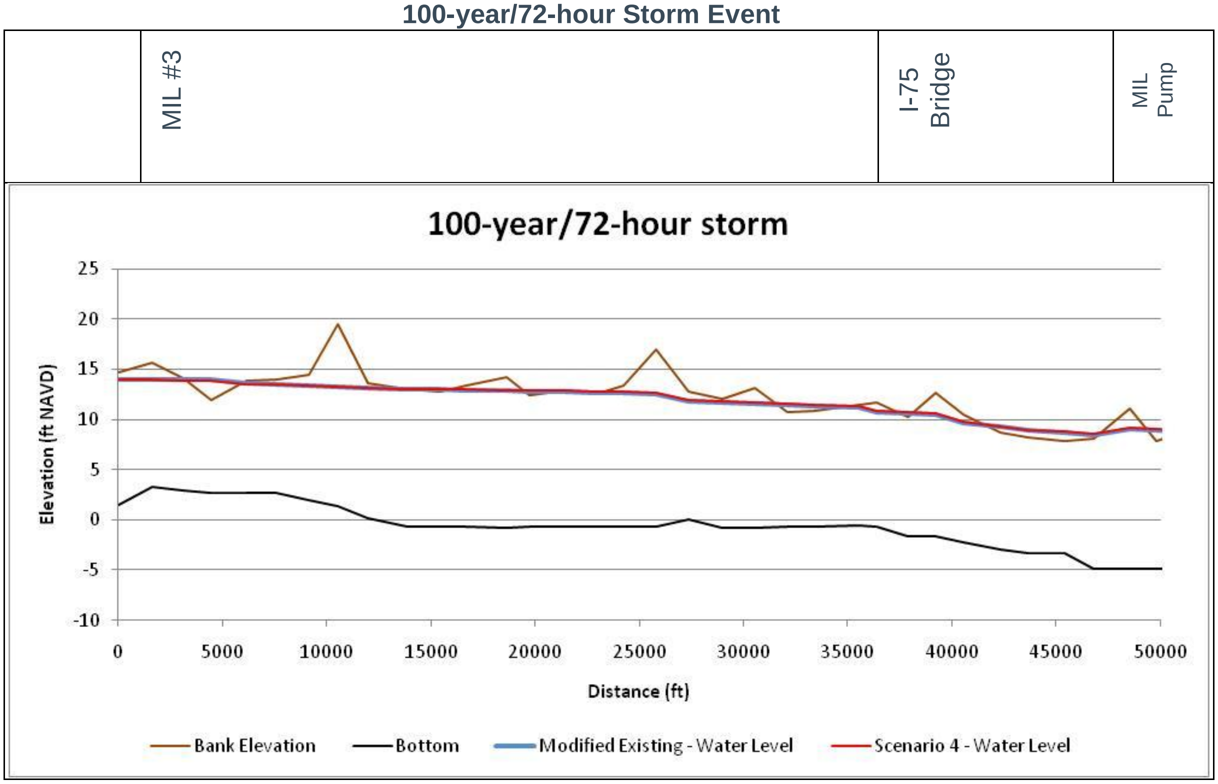


Figure 78. Surface Water Profile of Faka Union Canal 10-year/72-hour Storm Event

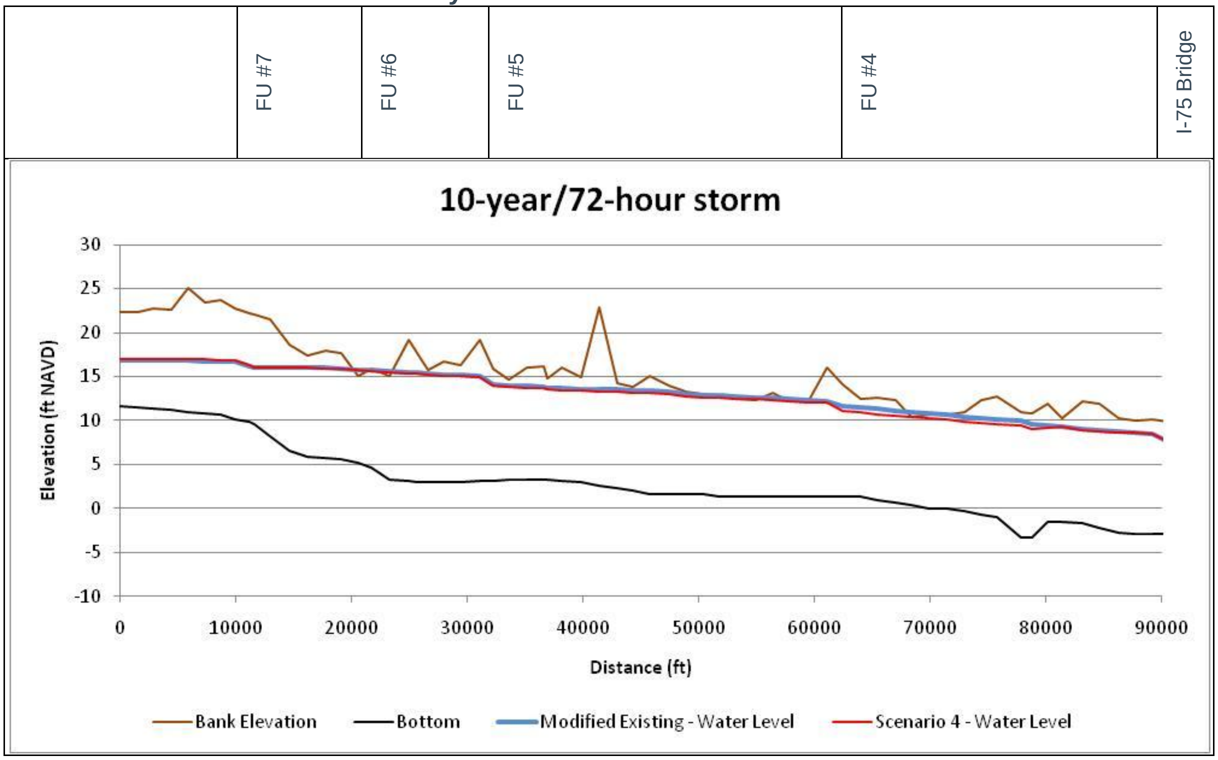


Figure 79. Surface Water Profile of Faka Union Canal 25-year/72-hour Storm Event

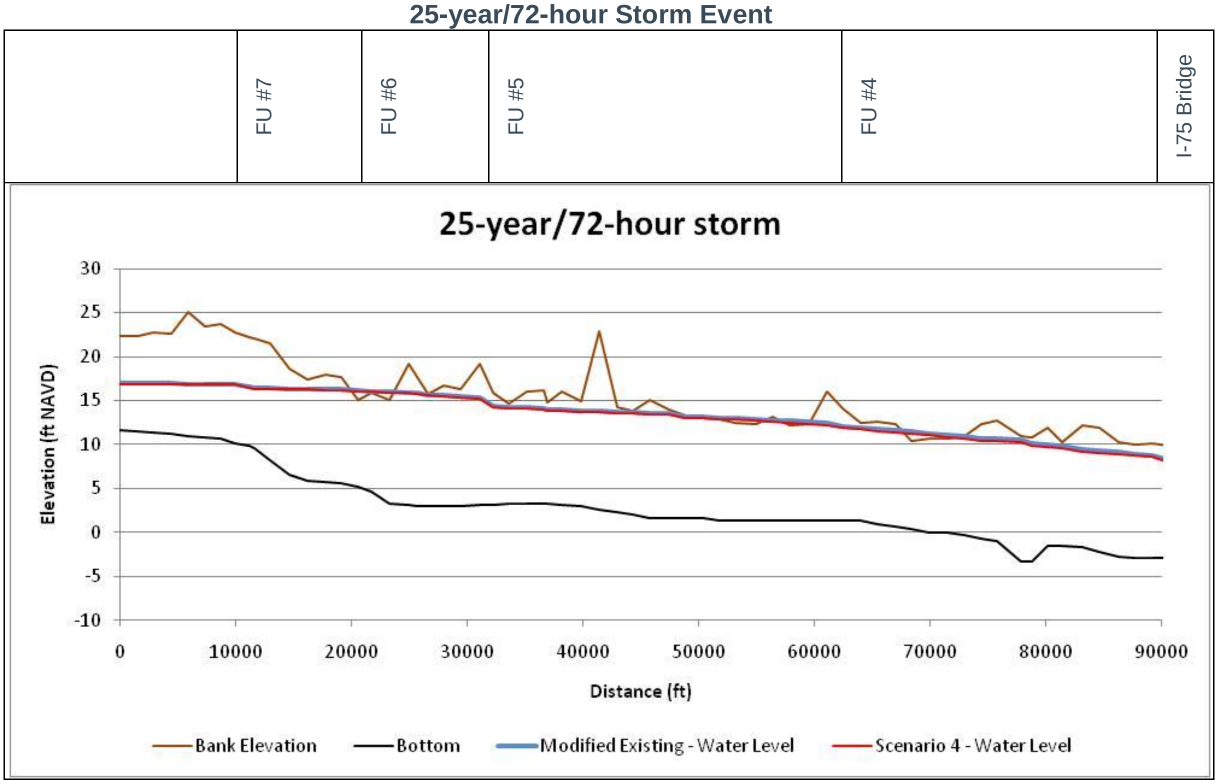


Figure 80. Surface Water Profile of Faka Union Canal 100-year/72-hour Storm Event

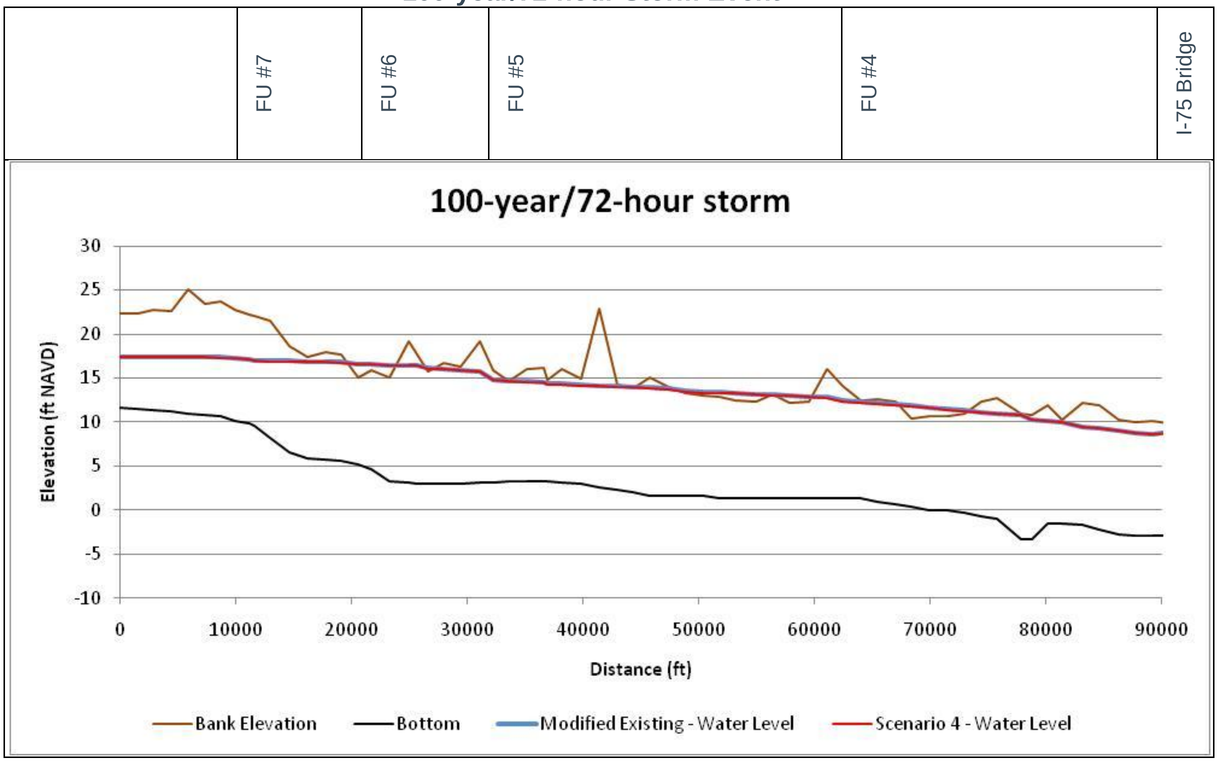


Figure 81. Scenario 4 - Difference in Depth of Storm Inundation; 10-yr/72-hr Storm

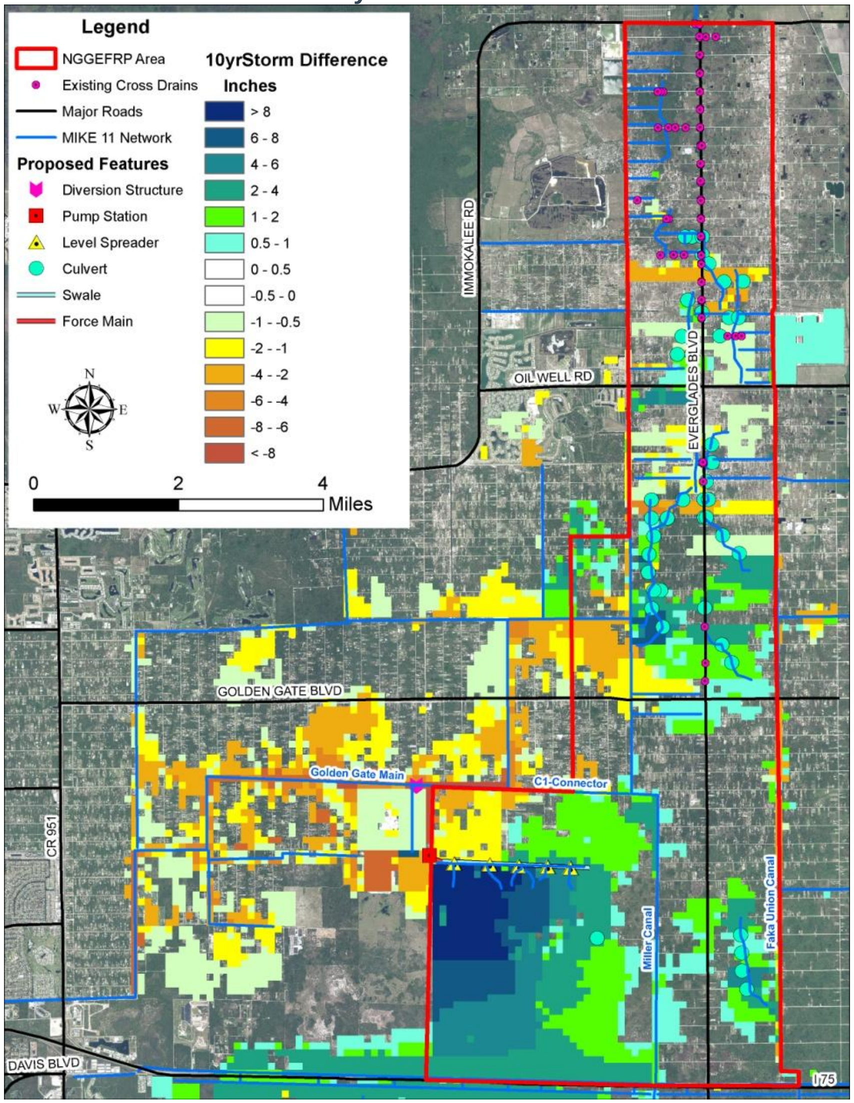


Figure 82. Scenario 4 - Difference in Depth of Storm Inundation; 25-yr/72-hr Storm

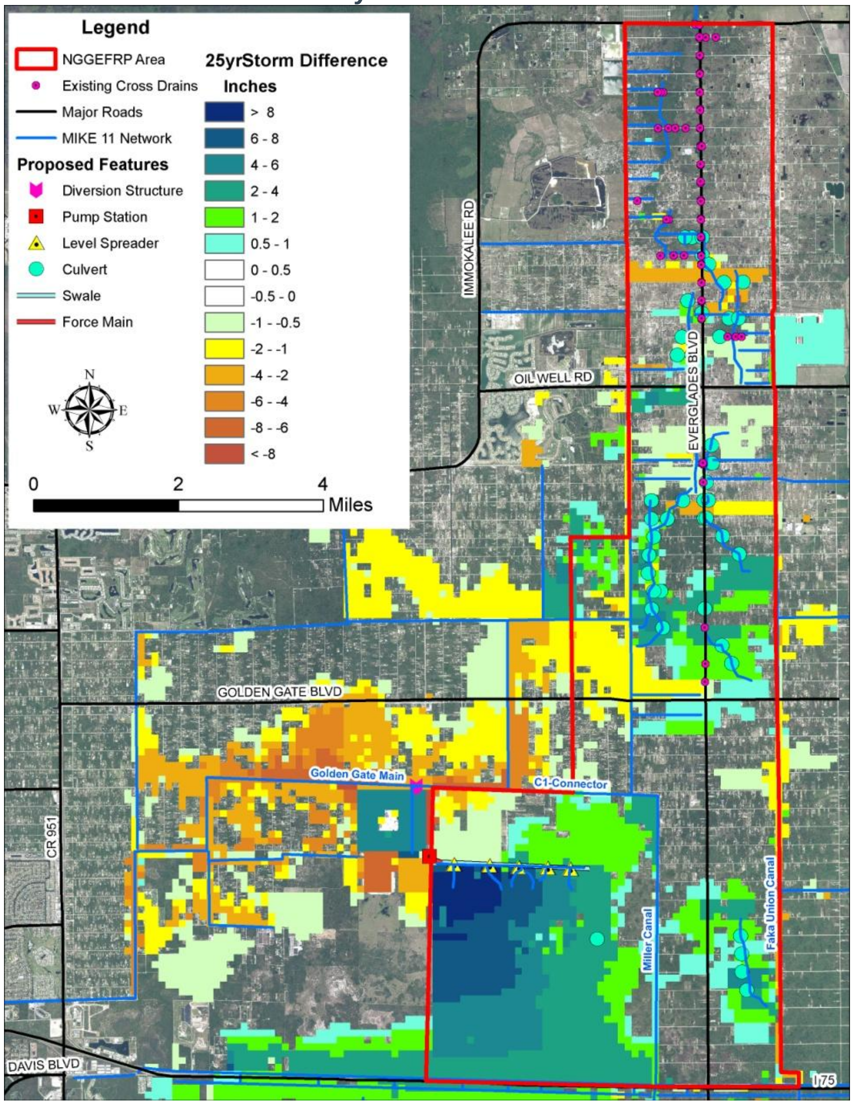
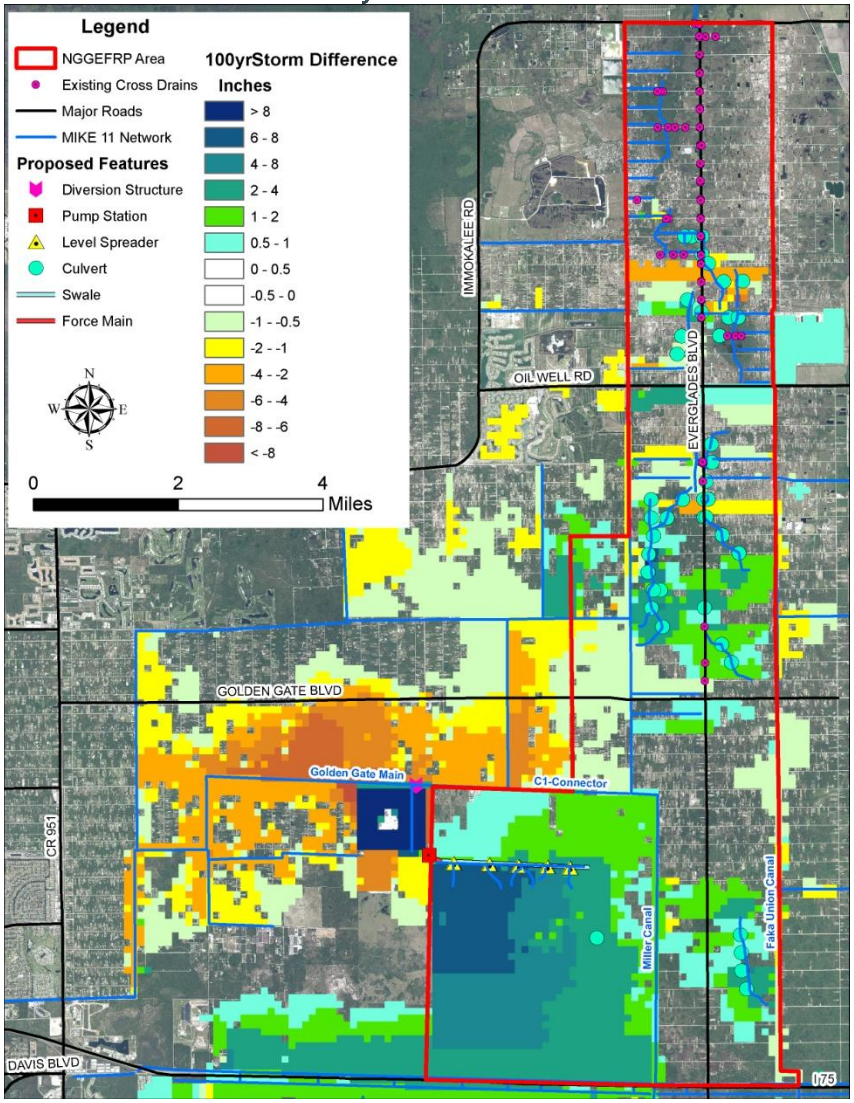


Figure 83. Scenario 4 - Difference in Depth of Storm Inundation; 100-yr/72-hr Storm



3.7.7. Predicted Change in Hydroperiod

A comparison of hydroperiod maps for Scenario 4 is shown in **Figure 84**. The results for the combined Scenario 4 are generally very similar to the individual maps presented for Scenarios 1 – 3. As in Scenario 1, Option 1, the most significant change in hydroperiod is found in the North Belle Meade area and results from the diversion of water from the Golden Gate Main Canal.

The most significant difference between the Scenario 4 hydroperiod results and the previous results occurs in locations where proposed culverts were removed from the model. This is most obvious in the Panther Walk area and the area immediately south of Oil Well Rd. In these locations the Scenario 4 result predicts little or no change from the Modified Existing Conditions Model.

3.7.8. Predicted Change in Wet Season Average and Seasonal High Water Elevation

The predicted difference in Wet Season average and Seasonal High Water Levels are shown in **Figures 85 and 86**. A positive value indicates that Scenario 4 water levels are higher; negative values mean that the Modified Existing Condition results are higher. These results are also very similar to results presented for Scenarios 1 – 3. The Wet Season Average groundwater head elevation map shows that groundwater elevations will be higher in Scenario 4 in the North Belle Mead area and in portions of the NGGE flowway between the Miller and Faka Union Canals. These results also indicate that groundwater flows to the south and west from the North Belle Meade area. If mines are constructed in that area in the future, those elevations will be affected.

The spreader swale appears to affect groundwater elevations in the area immediately north of the swale near the intersection of the Miller and C-1 Connector Canals. This may contribute to the predicted increase in the depth of inundation seen in **Figures 81 – 83**.

The seasonal high groundwater elevation near the canal network is directly affected by the water level in the canal. **Figure 67** indicated that the water surface is approximately six (6) inches lower in the Golden Gate Canal during the wet season. This directly contributes to the predicted decrease in groundwater elevations upstream of the GG-3 structure adjacent to the canal as shown in **Figure 86**. **Figure 72** indicates the water level in the Miller Canal increased as a result of inflows from the North Belle Meade area. This change in elevation is reflected in an increase in groundwater elevations adjacent to the Miller Canal.

As in Scenario 3, there is an increase in wet season average groundwater elevation and the seasonal high groundwater elevation near the intersection of the Golden Gate and Miller Canals west of Everglades Blvd that may affect septic systems in some homes in the area.

Figure 84. Scenario 4 - Hydroperiod Comparison:

Modified Existing – Predicted Hydroperiod | **Scenario 4 – Predicted Hydroperiod** | **Scenario 4 - Difference in Predicted Hydroperiod**

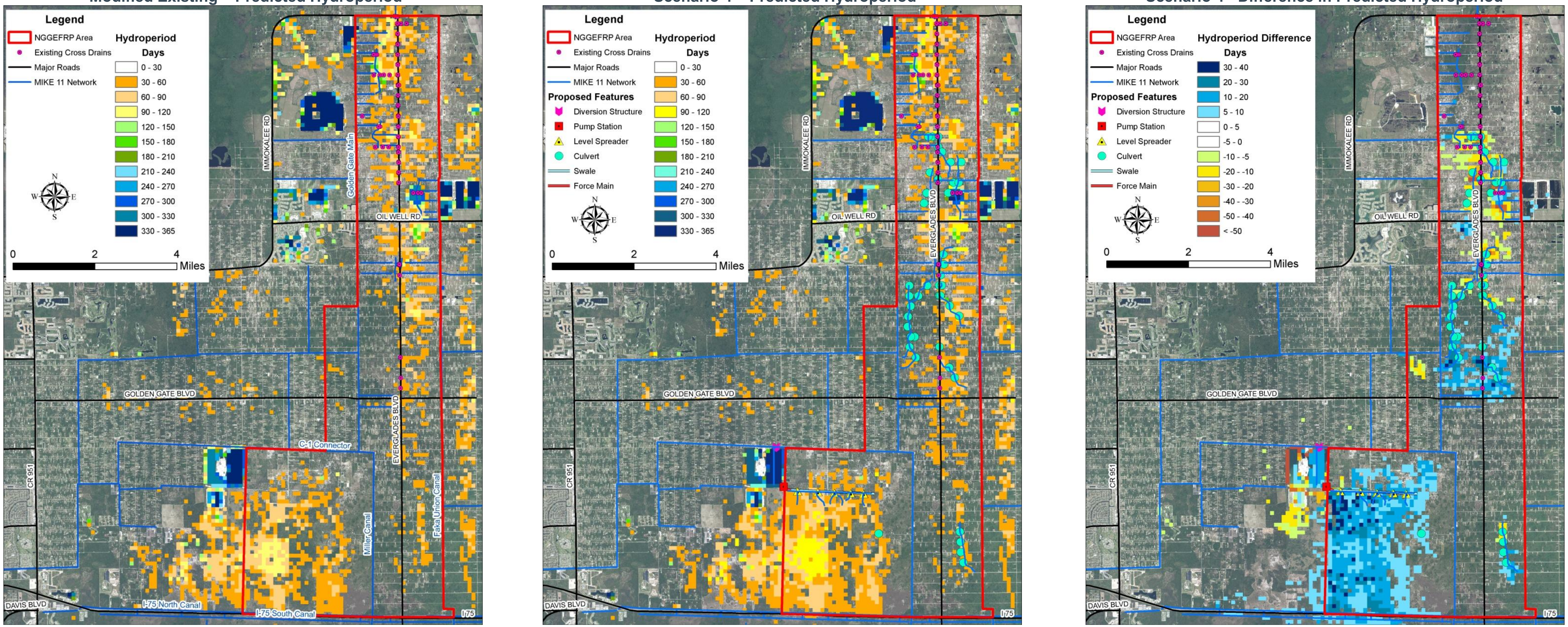


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Figure 85. Scenario 4 – Difference in Wet Season Average Groundwater Elevation

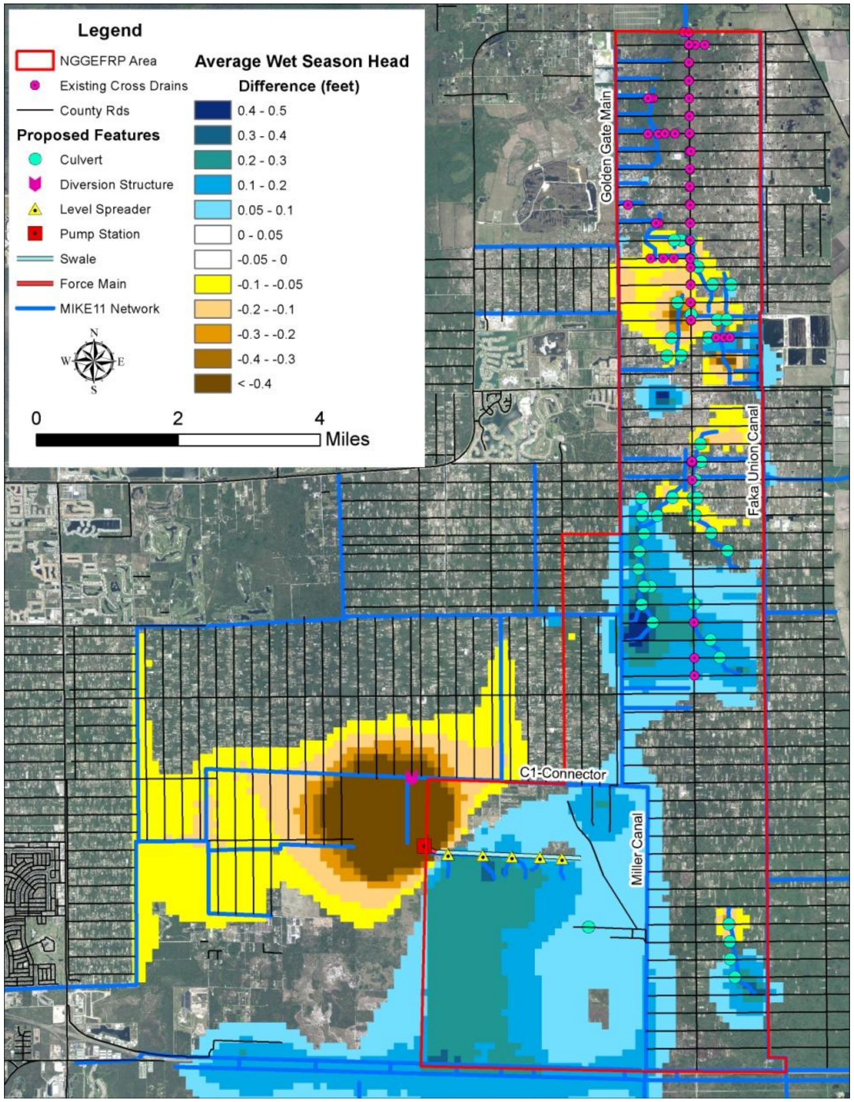
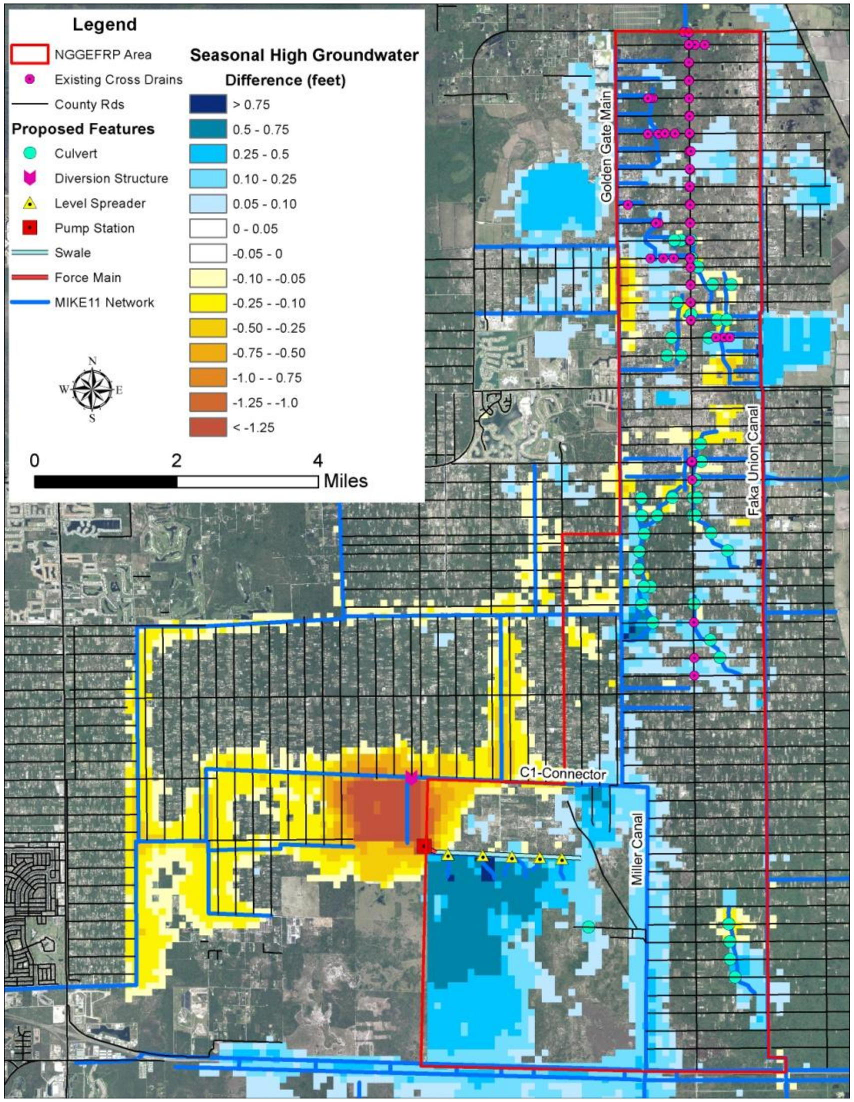


Figure 86. Scenario 4 – Difference in Seasonal High Groundwater Elevation



3.8. Scenario 4 – Recommendations

The following conclusions and recommendations were drawn from the analysis completed for Scenario 4.

- There is an overall improvement in the reduction of flows to Naples Bay resulting from the implementation of the North Belle Meade Spreader project.
- Increased flows to Henderson Creek resulting from the spreader system may need to be mitigated so that there is no increase in flows to Rookery Bay. The South I-75 Spreader Swale and the Henderson Creek off-line storage projects indentified in the CCWMP may be able to offset the additional flow.
- The increased flow to the Miller Canal resulting from the North Belle Meade Spreader does not impact the overall conveyance capacity or the drainage swales that discharge to the Miller Canal. The results indicate that the enhanced flowway will not adversely impact the delivery of water, or the overall goals of the PSRP.
- Adding cross-drains north of Oil Well Rd. may be detrimental to existing wetland areas. It is recommended that zero cross-drains be added in the Panther Walk area.
- Adding cross-drains south of Oil Well Rd. is predicted to increase groundwater elevations near the junction of the Golden Gate and Miller Canals. This may be beneficial by providing additional recharge to the County Well field.
- There is no perceived benefit to adding cross-drains in the area west of the Miller/Golden Gate Canal along 16th Street NE.
- Model results suggest that homes constructed on relatively low pads may have their septic leach fields affected by the change in the seasonal high groundwater elevation. This is particularly relevant near 8th Avenue NE, west of Everglades Blvd.

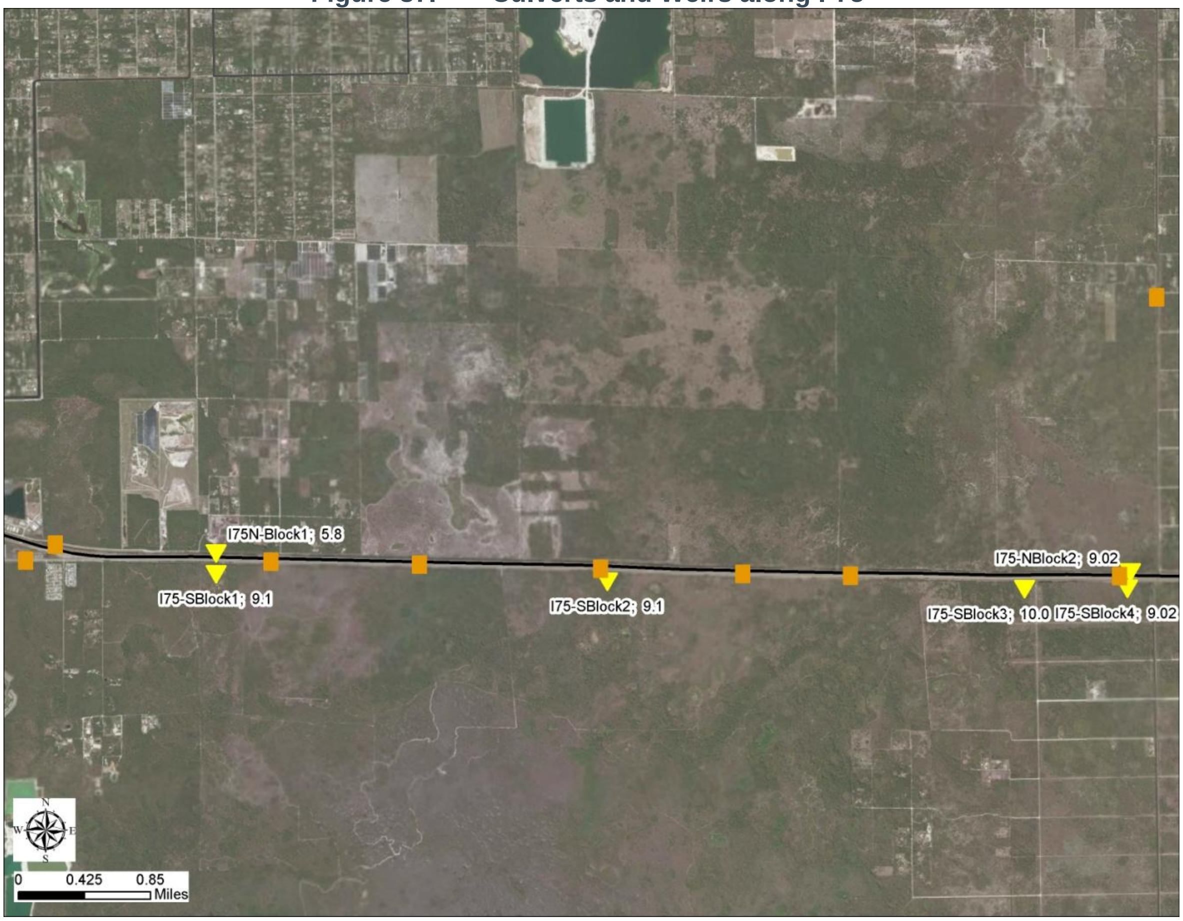
The results of the Scenario 4 analysis suggest that the project as a whole will partially meet the goals of diverting water from the Golden Gate Canal, increasing wetland habitat in the Golden Gate Estates, and providing additional recharge to the County wellfield along Everglades Blvd. It appears that several homes would be affected by increases in groundwater elevation. It may be possible to fit many of the culverts with operable flap gates or drop structures to minimize impacts on downstream private property. The County would have to consider appropriate actions to reduce the potential impacts on downstream property owners. Implementation of the proposed flowway features would also need to be incorporated into the water management features of future road improvements in the North Golden Gate Estates. These include roads like the Vanderbilt Beach Road extension, Randall Blvd improvements, and the extension of Wilson Blvd.

It is noted that the installation of cross-drains north of Oil Well Rd may have a detrimental effect on existing wetland areas and are likely to provide little benefit to the reduction of flows in the Golden Gate and Faka Union Canals.

The largest concern of the analysis is the excess flows leaving the North Belle Meade area and entering Henderson Creek. The additional flows may increase the wet season flow surplus to Rookery Bay identified in the Collier County Watershed Management Plan. Additional projects would likely be required to minimize potential impacts to Rookery Bay and the Ten Thousand Island estuaries. The use of a smaller diversion structure would reduce the risk to the Ten Thousand Islands Estuary, but would decrease the benefit to Naples Bay.

Implementation of the South I-75 spreader swale would likely reduce the volume of flow to Henderson Creek and the Miller Canal. The ground surface immediately south of I-75 is typically around 10.0 NAVD and the ditch blocks in the South I-75 canal are higher than those in the North I-75 canal (**Figure 87**). The result is that excess water preferentially flows to the east and west from the I-75 North canal to exit the North Belle Meade area. The South I-75 spreader swale as described in the CCWMP calls for a small pump station in the South I-75 canal to send water into existing wetland areas located further south. It may also be possible to raise the elevation of the existing ditch blocks on the north side of I-75 to an elevation of approximately 10.5 NAVD and add a new ditch block near the location of the proposed Wilson Blvd Extension. These improvements, coupled with increased elevations on the South I-75 ditch blocks may be sufficient to generate overland flow to the south from the South I-75 canal.

Figure 87. Culverts and Weirs along I-75



3.9. Preliminary Engineering and Preliminary Cost Estimates

Tables 22 and 23 provide preliminary cost estimates for construction of cross-drains required for the NGGE Flowway Restoration and the components of the North Belle Meade Spreader Swale. For the cross-drains, it was assumed that the required trench would be 30 – 36 inches deep at the pavement crossing and that the local water table is approximately 12 - 18 inches below grade; creating a situation where dewatering may be required. The dewatering would require excavation of a sump at each end of the pipe installation with a minimum of an earthen cofferdam around each sump to prevent the pumped water from re-entering the pipe trench. It was assumed that one-half of the pipes would require dewatering activities at a cost of $2,000 per installation.

Preliminary design drawing for the components required to implement these projects are found in Appendix C.

Table 22. Preliminary Cost Estimate for NGGE Flowway Restoration Project.

ESTIMATE ITEM No.	DESCRIPTION	UNIT	QUANTITY	UNIT PRICE	TOTAL AMOUNT
	PROJECT WORK ITEMS				
1	Mobilization/Demobilization (2%)	LS	1	-	$11,191
2	Maintenance of Traffic and Road Safety Signage (2%)	LS	1	-	$11,191
3	Silt Fence Staked	LF	4,200	$2	$8,400
4	Clearing and Grubbing	AC	3.0	$20,000	$60,000
5	Dewatering	LS	43	$2,000	$86,000
6	Ditch Excavation & Grading	CY	600	$15	$9,000
7	Unsuitable Excavation	CY	100	$15	$1,500
8	Stone Bedding (for Pipe, Structures, and Box Culvert)	CY	180	$200	$36,000
9	18" RCP	LF	440	$50	$22,000
10	18" RCP MES	EA	22	$1,200	$26,400
11	24" RCP	LF	1,260	$65	$81,900
12	24" RCP MES	EA	66	$1,400	$92,400
13	Sodding (Bahia)	SY	8,400	$3	$25,200
14	County Utility Adjustment	EA	4	$8,000	$32,000
15	Private Utility Adjustment Coordination	EA	8	$1,000	$8,000
16	County Road Pavement Replacement	SY	800	$65	$52,000
17	Stablized County Road Repair	SY	250	$15	$3,750
18	Miscellaneous Work	LS	1	$15,000	$15,000
19	**Subtotal**				**$581,932**
20	**Engineering and Permitting**	10%			**$58,193**
21	**Contingency**	15%			**$87,000**
22	**Bid, Payment and Performance Bonds**	2%			**$11,600**
23	**Construction Estimate Total**				**$739,000**

Notes:
(1) Construction Phase Services, Legal, Easement and Right of Way Acquisition Services are not included
(2) Costs are by project element for the recommended alternative

Table 23. Preliminary Cost Estimate for North Belle Meade Spreader Swale Project

PUMPING STATION & SPREADER SWALE PROJECT					
ESTIMATE ITEM No.	DESCRIPTION	UNIT	QUANTITY	UNIT PRICE	TOTAL AMOUNT
	PROJECT WORK ITEMS				
1	Mobilization/Demobilization (2%)	LS	1	-	$229,107
2	Soil Tracking Prevention	LS	1	$5,000	$5,000
3	Silt Fence Staked	LF	24,000	$2	$48,000
4	Hay or Straw, Bales	TN	5	$500	$2,500
5	Floating Turbidity Barrier	LF	160	$10	$1,600
6	Dust Control Measures	LS	1	$4,000	$4,000
7	Clearing and Grubbing	AC	80	$4,000	$320,000
8	Dewatering	LS	1	$25,000	$25,000
9	Regular Excavation	CY	200,000	$5	$1,000,000
10	Unsuitable Excavation	CY	100	$15	$1,500
11	Finish Grading	AC	48	$2,000	$96,000
12	Stone Bedding (for Pipe, Structures, and Box Culvert)	CY	75	$200	$15,000
13	Steel Sheet Piling Walls (PZ 11)	SF	6,000	$20	$120,000
14	Stone Riprap	CY	1,900	$200	$380,000
15	Soil Filter Cloth	SY	4,000	$5.00	$20,000
16	Concrete Retaining Wall Cap	CY	25	$600	$15,000
17	Steel Sheet Piling Walls (PZ 27)	SF	4,000	$40	$160,000
18	6'x4' RCB Culvert	LF	240	$525	$126,000
19	Concrete Endwalls	CY	55	$750	$41,250
20	Operable Gates	EA	6	$25,000	$150,000
21	SCADA Control System	EA	1	$100,000	$100,000
22	54" HDPE FM	LF	3,800	$225	$855,000
23	Pump Station (800 cfs)	LS	1	$7,000,000	$7,000,000
24	Driveway Pavement	SY	2,000	$60	$120,000
25	6' Height Chain Link Fence	LF	1,200	$20	$24,000
26	FDOT Type A Fence	LF	12,000	$7	$84,000
27	Sodding (Bahia)	SY	250,000	$3	$687,500
28	18" RCP	LF	40	$50	$2,000
29	18" MES	EA	2	$1,000	$2,000
30	Miscellaneous Work	LS	1	$50,000	$50,000
31	Subtotal				$11,684,457
32	Engineering and Permitting	10%			$1,168,446
33	Contingency	15%			$1,753,000
34	Bid, Payment and Performance Bonds	2%			$234,000
35	Construction Estimate Total				$14,840,000

Notes:

(1) Construction Phase Services, Legal, Easement and Right of Way Acquisition Services are not included

(2) Costs are by project element for the recommended alternative

3.10. Implementation Strategy

This section discusses the implementation strategy for the recommended alternative. For purposes of this discussion, the North Belle Meade Spreader Swale and the NGGE Flowway Restoration Components will be discussed separately.

3.10.1. RESTORE Funding

Funds may be available through the RESTORE Act to implement these projects. In general, the RESTORE Act will make funds available to Gulf Coast Counties through three mechanisms.

Direct to County – Thirty-five percent of the funds will be distributed directly to the states for distribution to the effected counties. It is expected that Florida will receive seven (7) percent of the available funds and that 25 percent of those funds would be distributed to the counties of southwest Florida. It is expected that that the funds received by each county will be used for environmental and economic restoration. It is anticipated that these funds would be used to support beach renourishment and other economic measures in the County.

Gulf Coast Ecosystem Restoration Council - The Act requires that 60 percent of the funds collected through the RESTORE Act be allocated to the Gulf Coast Ecosystem Restoration Council. Approximately half of these funds will be available for individual projects through a competitive process.

Consortia of Florida Counties – The final mechanism for distributing funds consists of a Consortia of 23 Florida Counties. This organization will develop a statewide plan and prioritize projects for implementation along the Gulf Coast. Collier County representatives are participating in this organization.

3.10.2. North Belle Meade Spreader Swale

The findings of this study indicate that the North Belle Meade Spreader Swale system will provide the benefit of reduced flows to the Naples Bay Estuary. However, the results also indicate that additional measures may be required to manage the water diverted into the Henderson Creek and Miller Canals.

The South I-75 and Henderson Creek off-line storage projects identified in the Collier County Watershed Management Plan may be appropriate to offset to increased flows to Henderson Creek. The effectiveness of the South I-75 spreader swale will be increased by modifying the existing ditch blocks in the I-75 canal.

It is also possible that the increased flows could be utilized by an increased demand for potable by the City of Marco Island.

The Rookery Bay National Estuary Research Reserve is currently selecting a consultant to conduct an evaluation of water management strategies in the Rookery Bay (Henderson Creek) watershed. This project provides an opportunity to evaluate alternative strategies to manage the increase in flows to Henderson Creek.

Several mechanisms may be appropriate to fund this project.

Public-Private Partnership. One possible source of funding is the owners of the proposed mining operations immediately west of the project area. The land owners own much of the land within the footprint of the restoration area and will be required to provide mitigation for the proposed mining operations. A partnership could be reached with the mine operators to implement the proposed diversion and spreader system.

Gulf Coast RESTORE Act Funding. Collier County included the North Belle Meade Spreader Swale project as one of several local projects identified during development of the southwest Florida RESTORE Act Regional Plan that may be funded through the Consortia of Florida Counties. It may also be possible to bid for funding from the Gulf Coast Restoration Council. Historically, the availability of matching funds, either through private partners, FDEP, or the SFWMD increases the opportunity to earn grant funding through a competitive process.

3.10.3. NGGE Flowway Restoration Project

The finding of this study indicate that the construction of cross-drains in the portion of the North Golden Gate Estates between the Golden Gate/Miller Canals and the Faka Union Canal will have little benefit in reducing flows to the canal network. The project may improve wetland habitat is some areas, but may potentially affect the septic drain fields of homes in these wetland areas.

Designation as a Wetland Mitigation Bank. The CCWMP recommended that this area be defined as a “sending” area under the County Transferrable Develop Rights Program. This would provide a mechanism to obtain vacant wetland parcels or homes in wetland areas that may be affected by changes in the hydroperiod of the depth of water. It was also recommended that this project be identified as a Wetland Mitigation Bank. This would allow the county to sell wetland credits internally to the transportation department or externally to developers to fund construction activities.

Gulf Coast RESTORE Act Funding. Collier County included the North Golden Gate Estates Flowway Restoration project as one of several local projects identified during development of the southwest Florida RESTORE Act Regional Plan.

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Appendices

Appendix A. Comparison of Initial Local Scale Model Results to the CC ECM

Appendix B. Individual Calibration Plots for Each Monitoring Station

Appendix C. Preliminary Design Drawings